PRESTRESSED GIRDERS

Introduction

The precast, pretensioned girders covered herein are used in unshored, composite action with the concrete deck. They are most commonly used in the **12 to 40 m** span ranges. The prestressed girders in present use are the standard AASHTO Types I, II, III, IV, IV modified, and the bulb tee (1830 BT).

Scope

This article is to supplement and amend the design presented in the AASHTO Specifications to conform to the design policy of the Department.

Commentary

The deck used is constructed exclusively of normal weight aggregate with normal strength structural concrete. The girder concrete is an early high strength concrete.

The girders are precast and pretensioned in manufacturing plants usually away from the job site. The girders are mass produced elements that are subject to a variety of different methods of fabrication depending on the individual manufacturers.

It is desirable to minimize the number of strand patterns in a project. Frequently, girders of similar length or loads can be grouped into a particular strand pattern.

Prestressed girders shall be made continuous for the maximum practical length, to eliminate expansion joints. The prestress girders shall be designed as simple span girders for positive moment, without regard to live load continuity. The prestress girders shall be designed to account for live load continuity for shear and negative moment design. The girders are assumed to be continuous because they are embedded into the continuity diaphragm that is poured in-place with the deck slab. Additional reinforcing steel shall be placed in the cast-in-place deck slab to resist the continuous live load negative moments developed as a result of the continuity.

Analysis

1. In the design analysis the initial camber (deflection at release due to prestress and self-weight) can be assumed to be the final camber (deflection after all dead loads and applied prestress). For construction purposes the initial camber and expected dead load deflection are calculated and given.

2. To relieve the stress at the ends of the girders, prestress girders will be designed utilizing either the debond or draped strand design. The preferred debond design must follow AASHTO LRFD limits on total percent debonded and percent debonded per row. If the debond design results in a more costly design (four or more additional strands per girder, additional girder line, etc.), then a draped pattern should be utilized.

3. The sidewalk and traffic railing dead and live load shall be assumed to be carried by the exterior girder. If in the judgement of the designer, the sidewalk and the parapet places an excessive load on the exterior girder, he may distribute a percentage of the load to the adjacent interior girder(s) as directed by the Bridge Design Engineer. Exterior girders shall not have less capacity than interior girders and will almost always be designed to carry the full weight of the barrier rail.

4. For design purposes, the haunch shall be taken as 15 mm less than the actual haunch dimension used at the centerline bearing to account for loss of structural depth due to camber.

5. It is desirable that the temporary tension stress in the top fiber near the ends of prestressed girders be reduced as much as possible by debonding (see #2 above and LRFD 5.11.4.2). If the tensile stress still exceeds the allowable after maximum debond, bonded non-prestressed reinforcing steel shall be supplied in an amount to resist the total tension force developed in the uncracked girder section at a working stress of 0.4 Fy. (See example, page 4-21 PCI Handbook, 4th edition).

6. A future wearing surface of 600 N/m^2 in dead load shall be added to the clear roadway width. However, no section loss or gain is assumed due to the wearing surface in the composite section.

7. Raised medians shall have required construction joints between the raised portion and the regular roadway slab. Girders affected by raised medians shall be designed to carry full live load & impact and dead load, exclusive of the weight at the raised portion, at 100% of the allowable stress. These girders affected shall also be checked with the previous loads plus the raised median superimposed for 150% of the Group I stress.

8. The design shall be made with 12.70 mm diameter 1860 MPa low relaxation (lo-lax) strands in accordance with ASTM A 416.

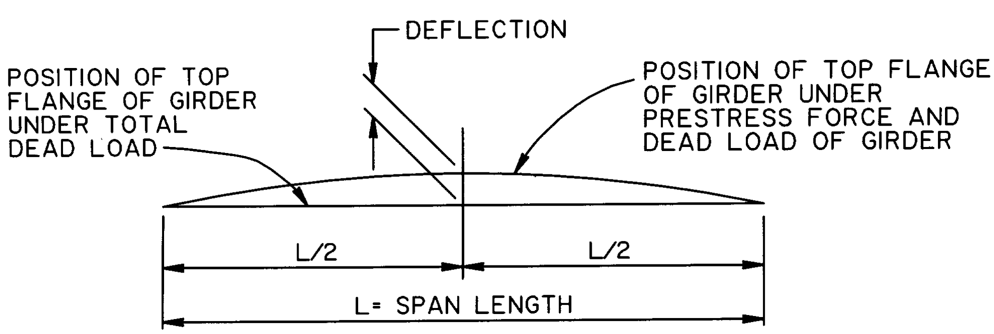
9. The actual camber (Δ) due to prestressing forces plus girder dead load may vary by **± 10 mm** from the predicted value, but the actual minimum camber should not be less than the predicted deflection due to composite and non-composite superimposed dead load (excluding future wearing surface). If the final deflection is negative, (i.e. sagging) then adding more strands or reducing the dead load to reduce the undesirable deflection will be required.

Let c = camber due to prestress force and girder weight
Δ = deflection due to diaphragms, deck, haunch, and barrier rail

Then at release: $c_{max} = c + 10$ mm
$c_{min} \geq (c - 10 \text{ mm}) \geq \Delta$

	EXT. GIRDER		INT. GIRDER	
DEAD LOAD REACTION	⊙		⊙	
DEFLECTION				
ALLOWABLE CAMBER	MIN	MAX	MIN	MAX

⊙ INCLUDES 600 N/m² FUTURE WEARING SURFACE.



DEFLECTION OF PRESTRESSED GIRDER UNDER DEAD LOAD OF CAST- IN- PLACE CONCRETE

10. Release and final strengths of concrete shall be **30 MPa** and **40 MPa** for **Class P**, and **35 MPa** and **45 MPa** for **Class P(M)**, respectively. **Class P(M)** concrete is to be used only with permission of the Bridge Design Section. The actual required design strength to the nearest 1 MPa, shall be the one shown on the plans.

11. Spans continuous for live load shall have additional reinforcing steel placed in the cast-in-place slab to carry the negative moment caused by live load plus impact at the continuous supports. Shrinkage and distribution reinforcing steel is to be included in the computation for determining the total amount of negative moment reinforcing required.

12. The shear requirements for prestressed girders shall be in accordance with the current AASHTO Specifications.

13. Girders subject to corrosion (road salt, gulf spray, etc..) shall incorporate means to reduce the potential of steel corrosion, such as calcium nitrite, microsilica, and fly ash.

14. Development length for fully bonded strands = 1.6 $\times$ [AASHTO Equation 9-32]
    Development length for debonded strands = 2 $\times$ [AASHTO Equation 9-32]

15. Prestressed girders that have a deck placed on them within a month or two of manufacture do not experience much camber or deflection growth. Therefore, a camber/deflection multiplier = 1.0 should be used on all standard projects.

16. Prior to final release of a project with prestressed girders, a final set of shop drawings (showing the strand type and pattern actually used in the bridge) shall be requested from the fabricator and transmitted to general files for microfilming.

17. Where PPCG are being used in an inefficient location (such as under a trapezoidal deck in a skewed span), it may be difficult to meet the M_{cr} requirement near the end ($^1/_{10}$ span point and outward) of the girder. Engineering judgment should be exercised in evaluating the specifications versus possible conditions.

Design Details

1. Girder spacing shall be equal where practical.

2. Generally, only exterior girders shall be anchored to the bent unless additional anchorage is required to meet seismic loading criteria. Spans subject to collision over navigable channels or subject to possible inundation by flooding shall be anchored at each girder.

3. The minimum haunch above the centerline bearing of the girder shall be **40 mm** for Type I, II and III girders and **50 mm** for Type IV, Type IV(mod), and **75 mm** for Type BT girders. For cases with high superelevation or a sag vertical curve, these may need to be increased so that there is no encroachment by the top girder flanges into the deck.

4. Girders with draped strands shall have tie-down points located symmetrically about the centerline of the span. The distance between the tie-down points shall be approximately $^{1}/_{5}$ to ¼ of the design span.

5. Diaphragm Policy:

   Spans ≤ **15 m** requires no intermediate diaphragm.
   15 m < spans ≤ **30 m** require one (1) intermediate diaphragm
   Spans > 30 m require two (2) intermediate diaphragms
   When no intermediate diaphragms are used, a note shall be placed in the plans requiring the contractor to provide temporary bracing during the deck pour. Intermediate diaphragms for skewed spans are usually constructed perpendicular to the girder webs by use of partial span width or stepping the diaphragms. For flared or skewed spans less than about 20°, diaphragms may be constructed at a skew to the web.

6. See details for standard end block for each girder type.

7. Neoprene bearing pads shall be in accordance with the miscellaneous span and girder details.

8. Precast-prestressed girder details shall be in accordance with the Miscellaneous Span and Girder Details sheets.

9. The maximum spacing of tie reinforcement shall meet AASHTO 9.20.4.5 as given in the table below.

AASHTO TYPE PPCG	MAXIMUM TIE SPACING(mm)	
	#10	#15
II	600	600
III	550	600
IV	450	600
BT	210	425

$$S \leq A \times F_Y \div 0.345 \times b_v$$

10. For prestressed girder projects in which the contractor elects to fabricate all the girders at the same time but girder placement will extend more than 4 months after casting (such as for phased construction or very large projects), the contractor will be required to account for camber growth. Camber for each girder shall be measured prior to erection and project engineer acceptable adjustments (such as lowering riser elevations) shall be made such that the top girder flange does not enter the bottom of the deck (maintain a positive haunch).

<u>Applicable Standard Drawings</u>

1. Miscellaneous Span and Girder Details (4 sheets).

PRESTRESSED GIRDERS WITH DEBONDED STRANDS

Introduction

The prestressed girders with debonded strands covered herein are intended to be used in lieu of girders with draped strands. This article is intended as a guide to the design of prestressed girders with debonded strands, and reflects current bridge design policy.

Commentary

Strands that are placed in a straight configuration near the bottom of a prestressed girder tend to cause overstress near the end of the girder at detensioning when the flexural capacity of the girder at midspan is efficiently utilized. To avoid this overstress, several strands centered in the web area are usually deflected or draped upward near each end to reduce the eccentricity of the force they cause about the centroid of the girder causing the overstress. The fabrication effort required getting the draped strands stressed and in position is time consuming, labor intensive, difficult and dangerous. It also requires the use of heavy equipment. This procedure contributes significantly to the fabrication cost of prestressed girders.

A less costly procedure is to place and stress the strands required for flexure at midspan in a straight configuration over the full length of the girder. The overstress near the ends of the girder at detensioning is avoided by debonding several strands from the ends of the girder towards midspan over a length which would otherwise be overstressed. This eliminates the eccentric forces the debonded strands would otherwise cause about the centroid of the girder and thus reduces the stresses.

The debonding of a strand is accomplished with a plastic tube shielding the strand from contact with the concrete mortar that bonds the strands to the concrete mass. This tube may be split with overlapping sides to facilitate installation along the strand as required and yet remain mortar tight. It must also have a thick and rigid enough wall to prevent collapsing on the strands. The end of the tube terminating in the concrete is taped closed to avoid concrete mortar seeping through. When the tube is successfully installed, none of the pretensioned force from the strand is transferred to the girder from its end to the point where the tube is terminated within the girder.

Since the unbonded prestressing steel and shielding material will remain in the girder, it is appropriate to minimize the use of debonding material and simultaneously attain the required flexural capacity of the girder to minimize cost. The strategy is to debond the strand from the ends only enough to eliminate overstress at detensioning and then check for adequate development lengths of the strands required to resist the factored moments at critical points.

The strands shall generally be debonded in a sequence beginning in the row nearest the bottom flange and progressing upward as required to satisfy design conditions.

Analysis

1. Determine the number of strands required at the midspan of the girder based on conventional procedures for service load and load factor analysis. Stresses and moments shall be determined at increments of 1/10 of the design span length or less.

2. Determine the number and location of the strands required to be debonded to avoid overstress at the ends of the girder at detensioning (initial conditions). The allowable tension shall be $\mathbf{0.249(f'_{ci})^{1/2}}$ and the allowable compression shall be $\mathbf{0.6f'_{ci}}$. For calculation purposes, the strands are assumed to introduce prestress force to the girder as follows:

   a) Transfer length for service load (initial and final conditions): Linearly from no contribution at the debond point to full effect over a distance of fifty times the strand diameter[4].

   b) Development length for ultimate strength: Linearly from no contribution at the debond point to full effect over a distance of two times the development length specified in the AASHTO Specifications subsection 9.28.1.

3. Establish the number and sequence of each set of strands to be debonded at intermediate points along the girder based on stresses at detensioning. Preferably, a 2 m distance between debond lengths will be utilized. Limit debond to 25% of total, 40% of any row (LRFD 5.11.4.2).

4. Exterior strands in each horizontal row should be fully bonded.

5. Locate the point that the strand pattern required at midspan will produce the maximum allowable stress in the girder at detensioning. Debond the first set of strands from this point to the end of the girder.

6. Locate the point that the remaining strand pattern will produce the maximum allowable stress in the girder at detensioning. Debond the second set of strands from this point to the end of the girder. Repeat this step for the third and additional sets of strands to be debonded at intermediate points along the girder.

7. Check stresses at final conditions at the centerline of the girder, at debond points and along the transfer lengths.

8. To satisfy the ultimate strength requirements, check that the strands bonded to the end of the girder are capable of resisting the factored moment at one development

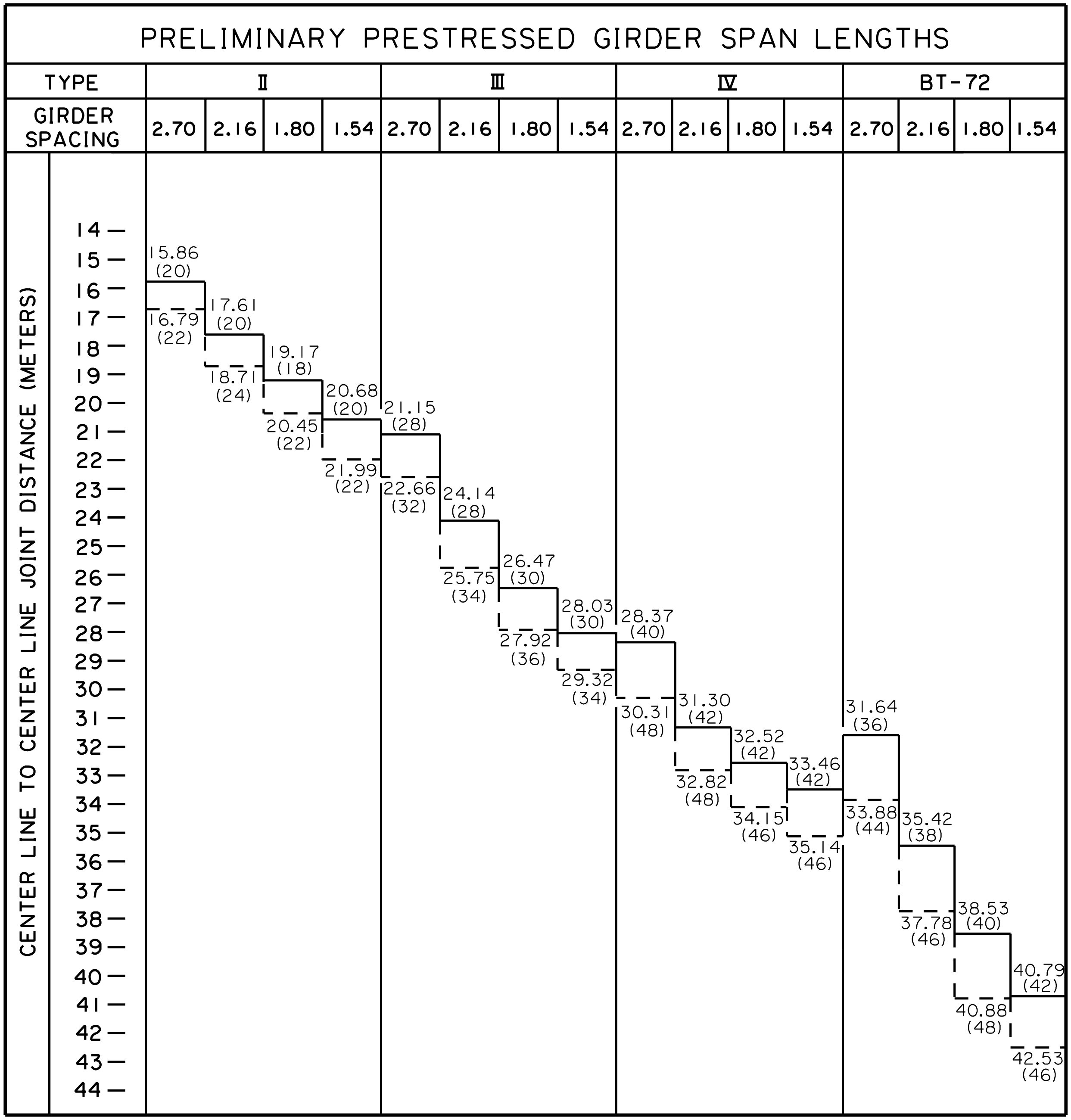
[4] Standard Specifications For Highway Bridges, Sixteenth Edition-1996, subsection 9.20.2.4

length from the ends of the girder. Then, check the capacity at all critical points, debond points and midspan, to insure that the ultimate moment capacity is greater than the factored moment. The ultimate moment capacity shall be the sum of the capacities of all fully developed strands plus the capacity of partially developed strands. At two development lengths from debond points, strands are fully developed. At points less than two development lengths from the debond point, the partial capacity of debonded strands shall be calculated based on a linear proportion from zero at the debond point to full capacity at two development lengths.

9. In the event that the number of strands required for the factored moment at a critical point cannot be bonded for two development lengths, determine the service load stress in the bottom fibers for Group I loads. If no tension stress occurs along the beam, one development length can be used in lieu of two development lengths.

Design Details

1. The use of debonding material and the number of debonding termination points should be minimized.



ASSUMPTIONS

1. 200 mm SLAB @ 24 MPa
2. 230 mm BEARING DISTANCE
3. HS 20-44(M) AND HST-18(M) LIVE LOAD
4. INTERIOR GIRDER SHOWN
5. BARRIER CARRIED BY EXTERIOR GIRDER EXCEPT FOR BT-72
6. 12.7 mm 1860 MPa STRANDS

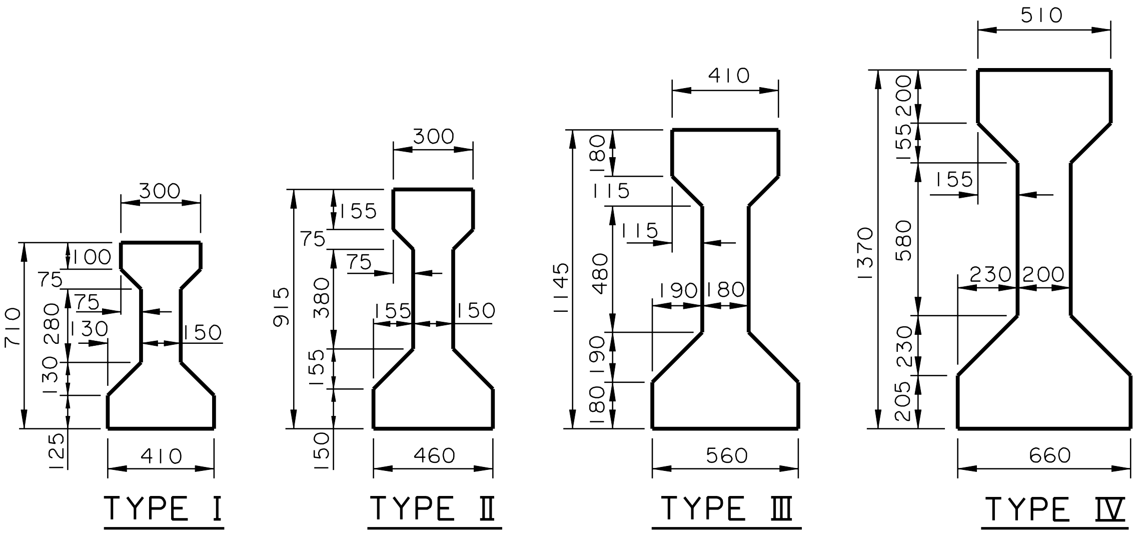
——— f'_c =40 MPa

– – – f'_c =45 MPa

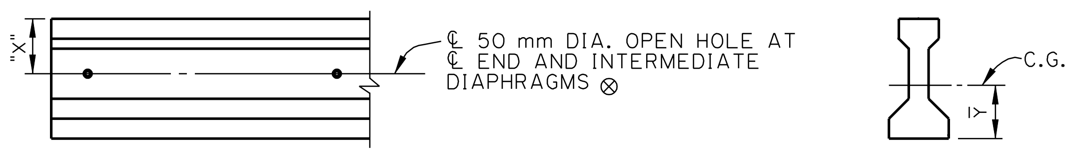
(xx) NUMBER OF STRANDS

APPROXIMATE SPAN LIMITS

CHART OF SPAN RANGE LIMIT FOR PRECAST-PRESTRESSED GIRDERS



BEAM PROPERTIES



BEAM TYPE	AREA (mm^2)	"X" * (mm)	$\bar{Y}$ (mm)	MOMENT OF INERTIA ($mm^4 \times 10^4$)	WEIGHT (kg/m) ❷	BOTTOM SECT. MOD. ($mm^3 \times 10^3$)	TOP SECT. MOD. ($mm^3 \times 10^3$)
I	176 525	315	316.52	934 754	423.66	29 532	23 756
II	236 650	420	400.30	2 113 520	567.96	52 798	41 063
III	365 225	535	517.12	5 290 770	876.54	102 312	84 264
IV	507 225	645	626.67	10 818 620	1217.0	172 636	145 543

* THIS DIMENSION MAY BE ADJUSTED TO CLEAR DRAPED STRANDS WHEN NECESSARY.
THIS DIMENSION IS TYPICAL FOR ALL GIRDERS REGARDLESS OF ROADWAY CROWN.

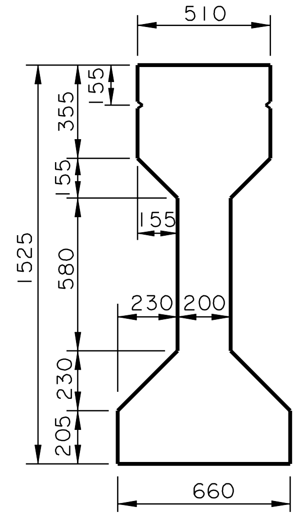
❷ BASED ON CONCRETE DENSITY OF 2400 kg/m^3

⊗ INTERIOR GIRDERS ONLY

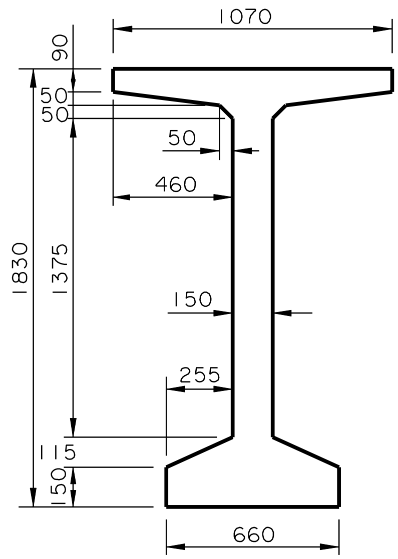
DIMENSIONS AND PROPERTIES

PRESTRESSED GIRDERS

1 OF 2

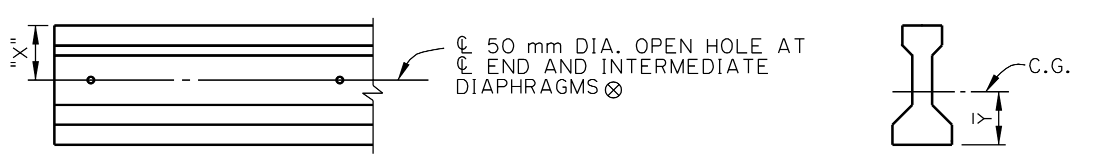


TYPE IV-MODIFIED



TYPE BT

BEAM PROPERTIES



BEAM TYPE	AREA (mm^2)	"X" ∗ (mm)	$\bar{Y}$ (mm)	MOMENT OF INERTIA ($mm^4 \times 10^4$)	WEIGHT (kg/m) ❸	BOTTOM SECT. MOD. ($mm^3 \times 10^3$)	TOP SECT. MOD. ($mm^3 \times 10^3$)
IV-MOD.	586 275	800	737.35	15 442 370	1408	209 431	196 056
BT	491 125	880	933.19	22 683 580	1179	243 076	252 936

❸ BASED ON CONCRETE DENSITY OF 2400 kg/m^3

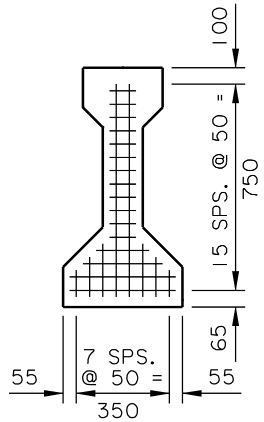
∗ THIS DIMENSION MAY BE ADJUSTED TO CLEAR DRAPED STRANDS WHEN NECESSARY. THIS DIMENSION IS TYPICAL FOR ALL GIRDERS REGARDLESS OF ROADWAY CROWN.

⊗ INTERIOR GIRDERS ONLY

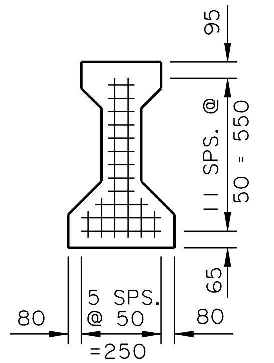
DIMENSIONS AND PROPERTIES

PRESTRESSED GIRDERS

2 OF 2



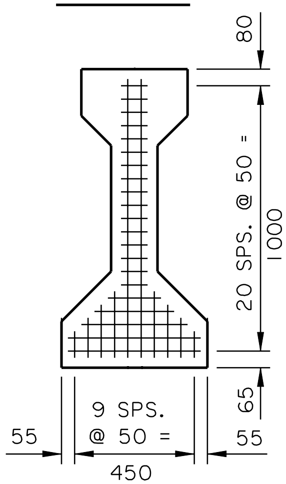
TYPE II



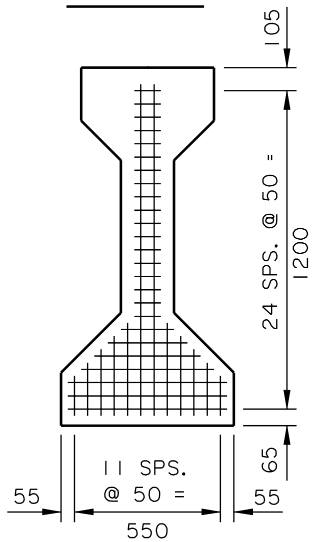
TYPE I

NOTE:

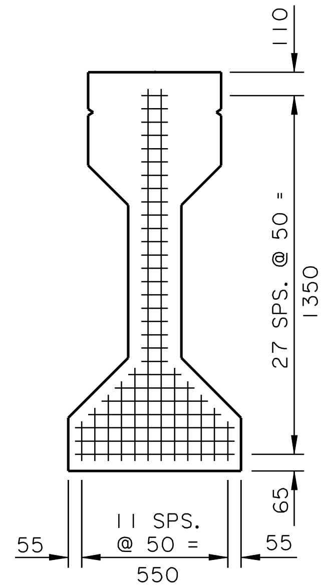
STRANDS ARE LOCATED ON A GRID
50 mm BY 50 mm STARTING 65 mm
FROM THE BOTTOM FOR ALL GIRDERS.



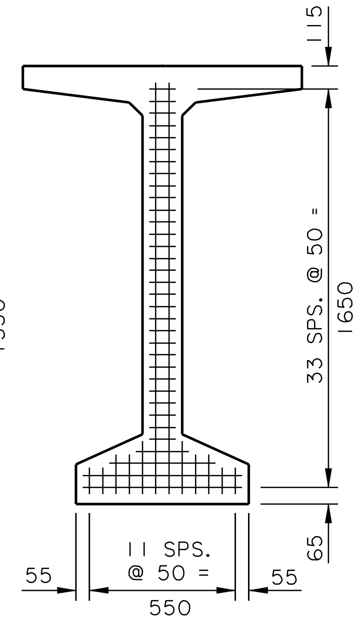
TYPE III



TYPE IV



TYPE IV-MOD.



TYPE BT

STRAND PATTERN TEMPLATES

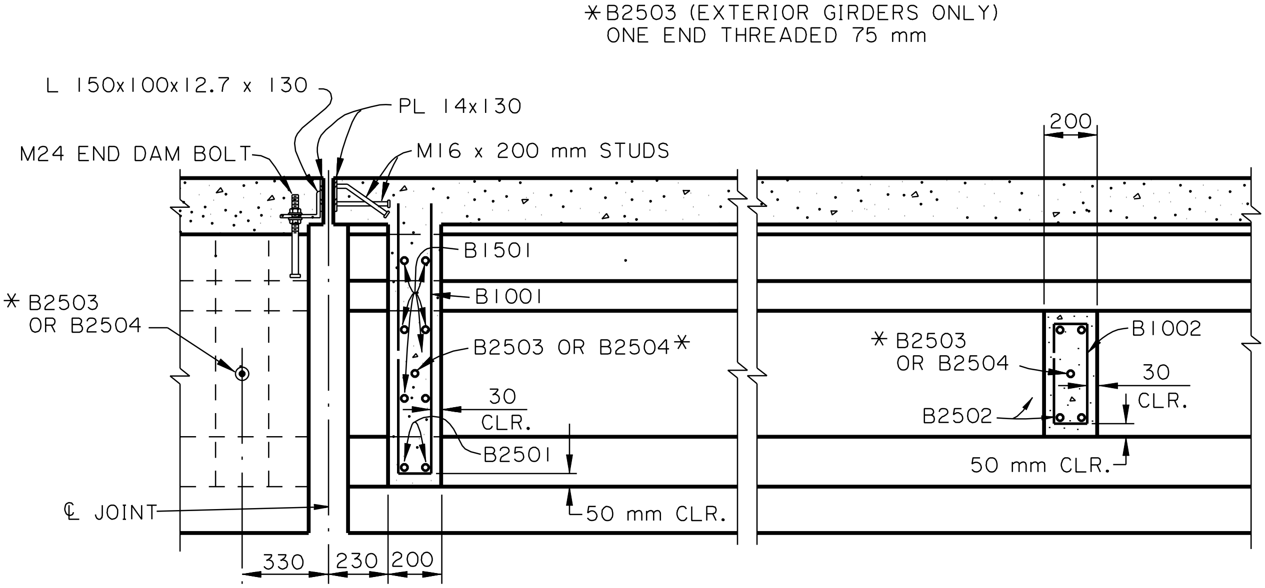
PRESTRESSED GIRDERS

Strand Properties Table

ASTM DESIGNATION A 416 UNCOATED 7-WIRE STRANDS FOR PRESTRESSED GIRDERS								
Strand Size (Dia)	Strand Grade (MPa)	Breaking Strength (kN)	Nominal Area (mm^2)	Weight (kg/ 1000 m)	Initial Load (kN) ⊙	Load @ 1% Extension (kN)	Initial Tension (70% Breaking Strength) (kN)	Initial Tension (75% Breaking Strength) (kN)
9.5 mm	1725	89.0	51.61	405	8.9	75.6	62.3	-----
	1860 S.R.	102.3	54.84	432	8.9	87.0	71.6	-----
	1860 L.R.	102.3	54.84	432	8.9	92.1	-----	76.7
11.1 mm	1725	120.1	69.68	548	12.0	102.3	84.1	-----
	1860 S.R.	137.9	74.19	582	13.8	117.2	96.5	-----
	1860 L.R.	137.9	74.19	582	13.8	124.1	-----	103.4
12.7 mm	1725	160.1	92.90	730	16.0	136.2	112.1	-----
	1860 S.R.	183.7	98.71	775	18.4	156.1	128.6	-----
	1860 L.R.	183.7	98.71	775	18.4	165.3	-----	137.8

S.R. DENOTES STRESS-RELIEVED.
L.R. DENOTES LOW-RELAXATION.

⊙ INITIAL TENSION PRIOR TO RELEASE OF STRANDS.



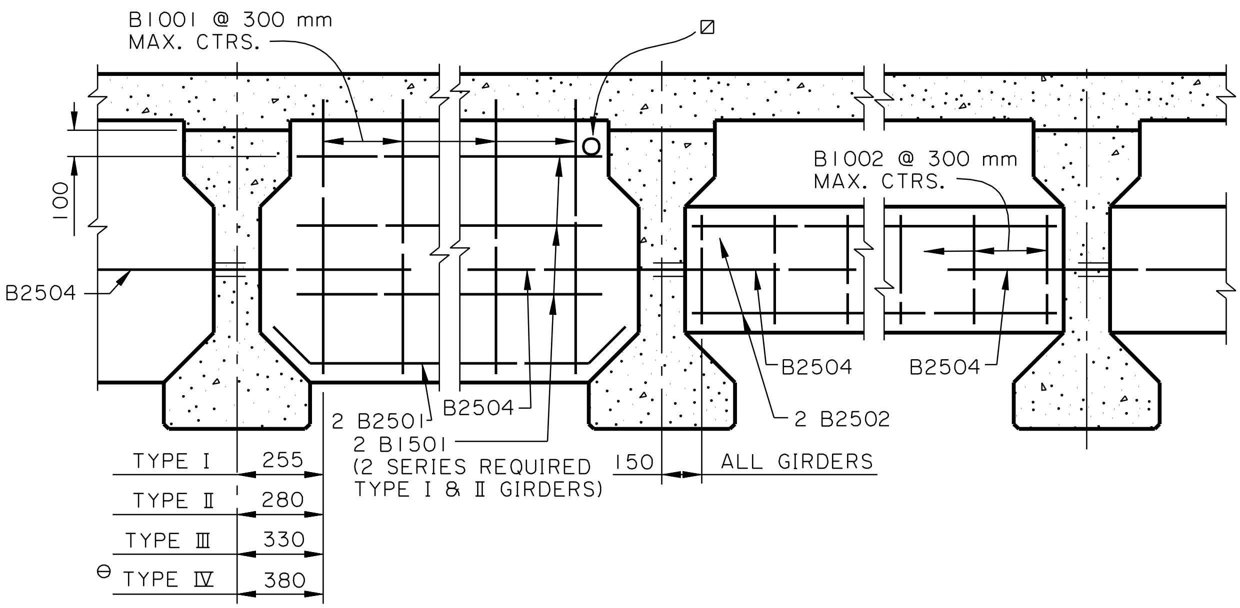
SECTION AT GIRDERS

SECTION BETWEEN GIRDERS

SECTION BETWEEN GIRDERS

END DIAPHRAGM

INTERMEDIATE DIAPHRAGM



END DIAPHRAGM

INTERMEDIATE DIAPHRAGM

⊖ INCLUDES TYPE IV-MODIFIED

NOTES:

SPANS >15 m REQUIRE ONE INTERMEDIATE DIAPHRAGM.

SPANS >30 m REQUIRE TWO INTERMEDIATE DIAPHRAGMS.

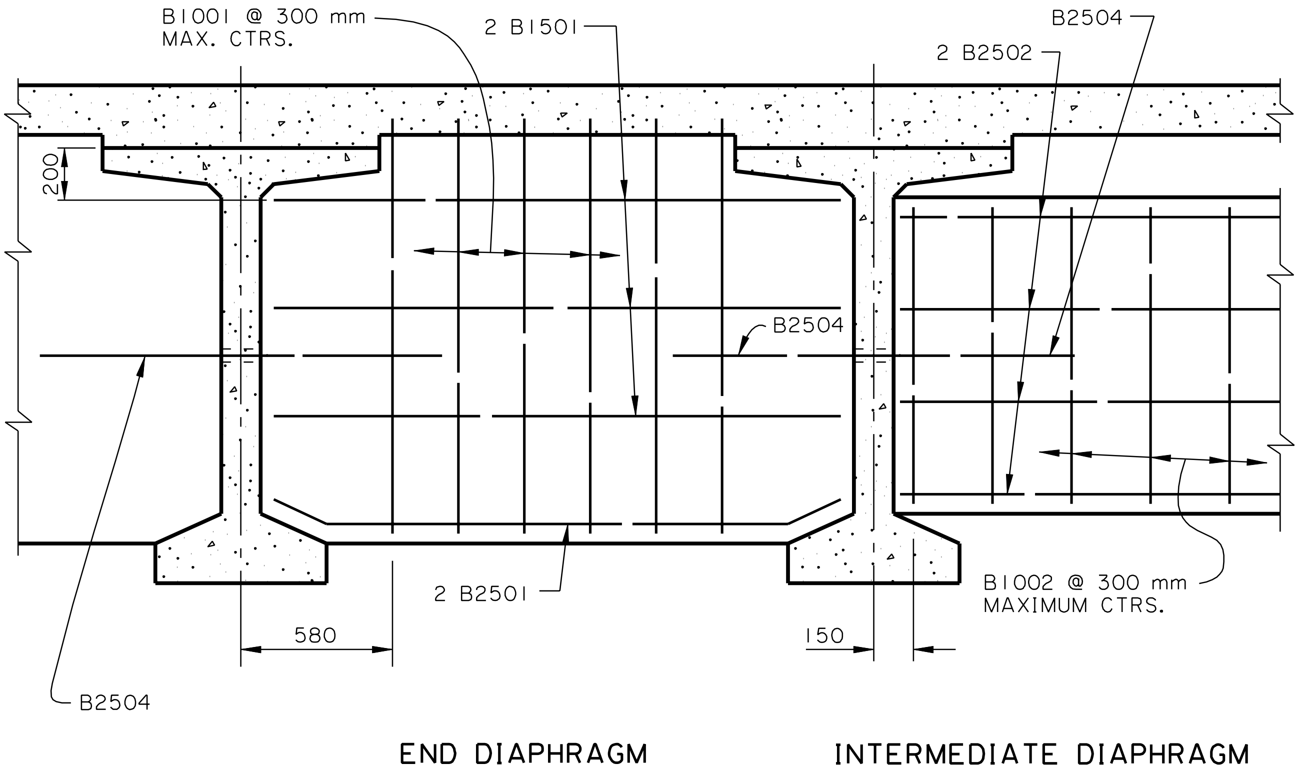
⊠ 75 mm Φ VENT HOLE IN EACH END DIAPHRAGM
WHERE INUNDATION BY FLOOD IS POSSIBLE.

TYPE I-IV (END & INTERMEDIATE DIAPHRAGMS)

DIAPHRAGM DETAILS

Rev. 04/01/99

1 OF 8



TYPICAL DIAPHRAGM DETAILS
(TYPE BT)

NOTE:

DETAILS NOT SHOWN ARE SIMILAR TO DETAILS SHOWN ON SHEET 1 OF 8.

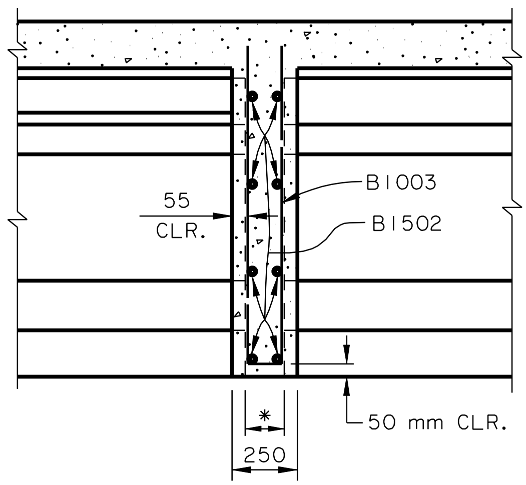
TYPE BT (END & INTERMEDIATE DIAPHRAGMS)

DIAPHRAGM DETAILS

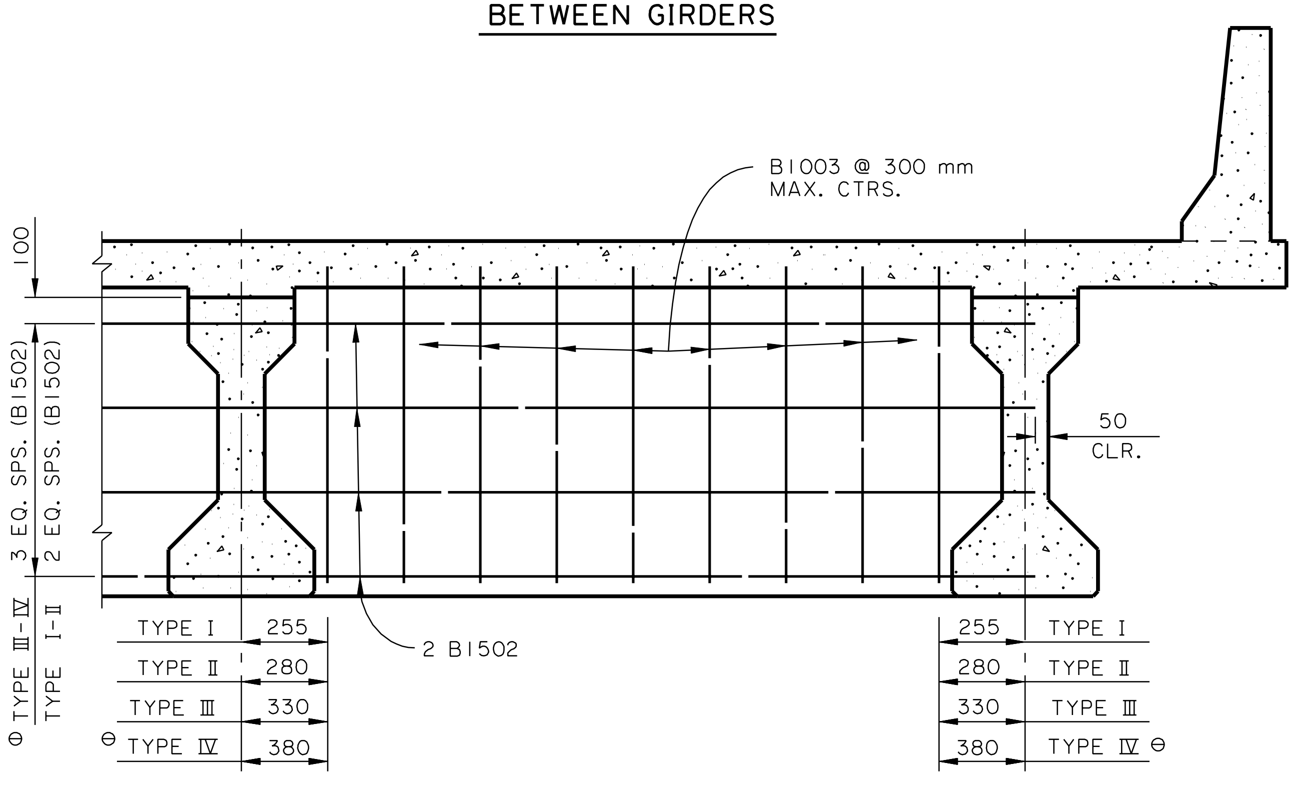
Rev. 04/01/99

2 OF 8

* 150 mm BETWEEN GIRDER ENDS



SECTION
BETWEEN GIRDERS



⊖ INCLUDES TYPE IV-MODIFIED

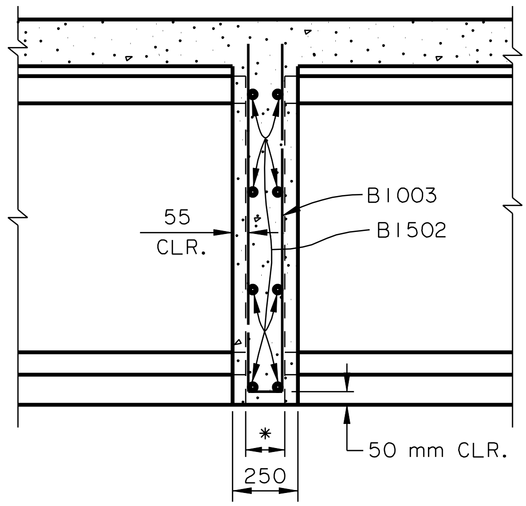
CONTINUITY DIAPHRAGM

TYPE I-IV (CONTINUITY DIAPHRAGMS)

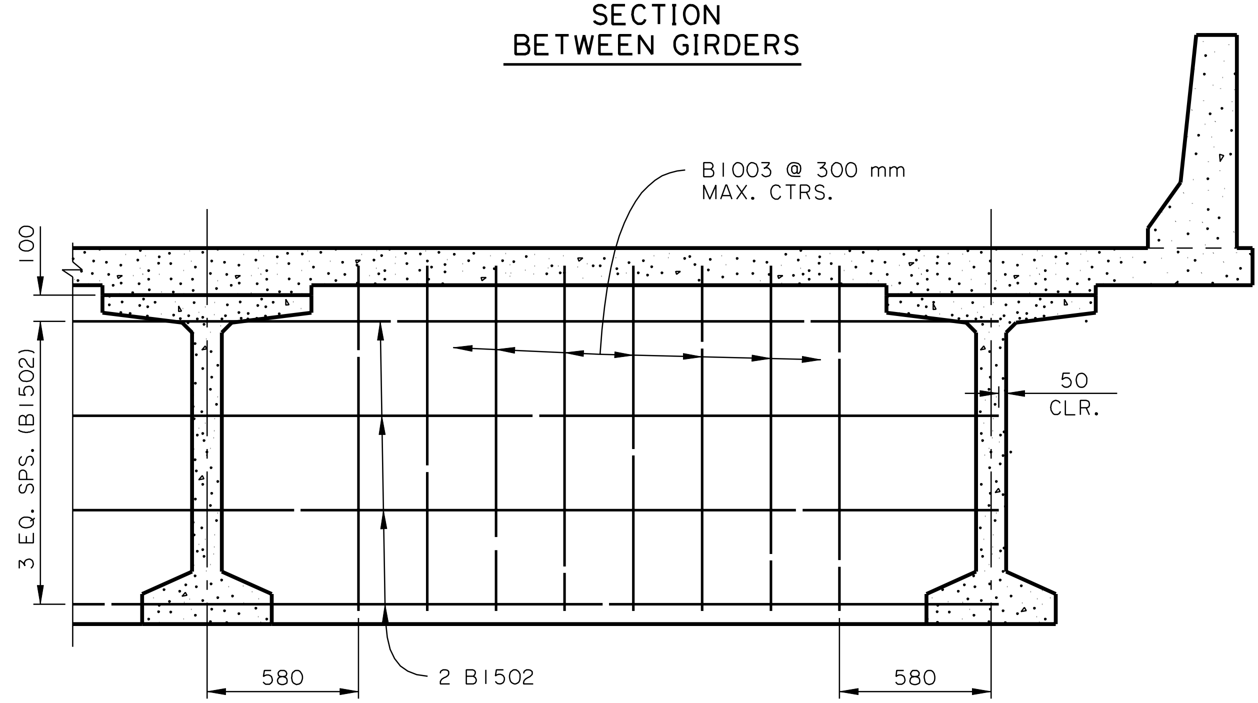
DIAPHRAGM DETAILS

Rev. 04/01/99

* 150 mm BETWEEN GIRDER ENDS



SECTION
BETWEEN GIRDERS



CONTINUITY DIAPHRAGM
(TYPE BT)

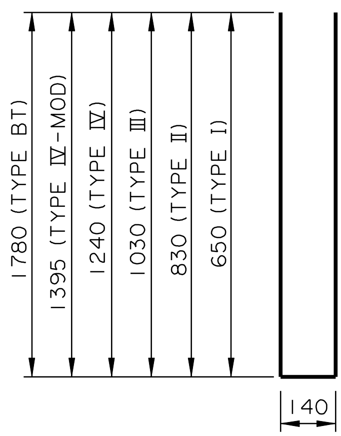
TYPE BT (CONTINUITY DIAPHRAGMS)

DIAPHRAGM DETAILS

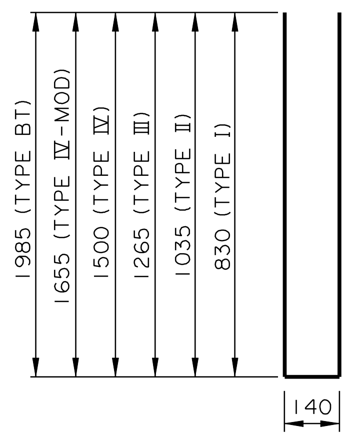
4 OF 8

Rev. 04/01/99

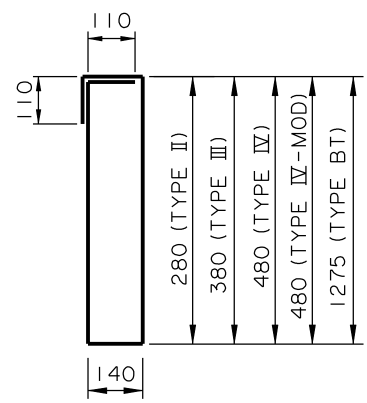
TOTAL LENGTH OF BARS						
BAR	TYPE I	TYPE II	TYPE III	TYPE IV	TYPE IV-MOD.	TYPE BT
B1001	1440	1800	2200	2620	2930	3700
B1002	N/A	1060	1260	1460	1460	3050
B1003	1800	2210	2670	3140	3450	4110
B1501	"Z"	"Z"	"Z"	"Z"	"Z"	"Y"
B2501	"X"+220	"X"+280	"X"+380	"X"+500	"X"+500	"X"+440
B2502	"Y"	"Y"	"Y"	"Y"	"Y"	"Y"
B2503	960	960	960	960	960	960
B2504	1980	1980	1980	1980	1980	1980



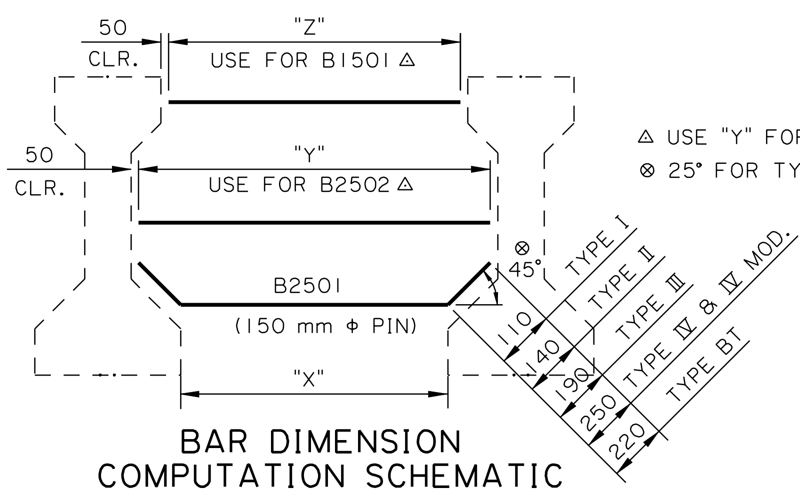
B1001
(45 mm φ PIN)



B1003
(45 mm φ PIN)



B1002
(45 mm φ PIN)



△ USE "Y" FOR B1501 (TYPE BT).
⊗ 25° FOR TYPE BT

BAR DIMENSION COMPUTATION SCHEMATIC (B1501, B2501, B2502)

TYPE I-IV AND BT (REINFORCING STEEL)

DIAPHRAGM DETAILS

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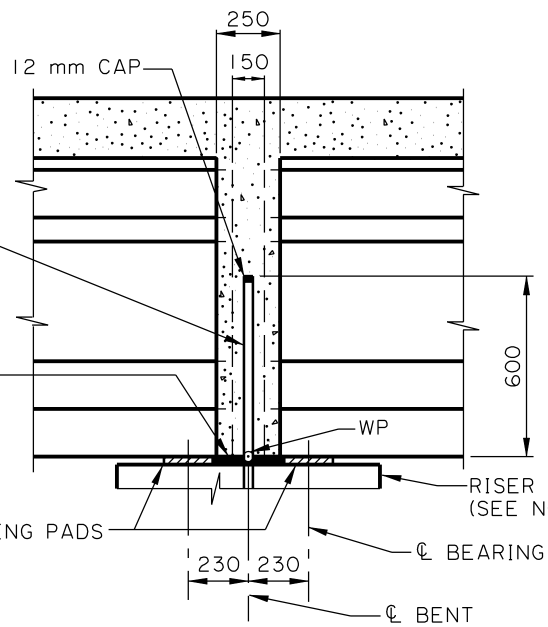

CONNECTION DETAIL AT CONTINUITY DIAPHRAGM

NOTE A

DOWELS IN THE CAP SHALL BE GALVANIZED NO. 35 DEFORMED REINFORCING STEEL. PRIOR TO POURING THE CONTINUITY DIAPHRAGM, EACH DOWEL SHALL BE SHEATHED BY 45 mm I.D. RIGID SLEEVE WITH A 12 mm THICK COMPRESSIBLE CAP ABOVE THE TOP OF THE DOWEL TO ALLOW THE GIRDERS TO DEFLECT ON THEIR BEARING PADS UNDER FUTURE LOADS.

TO FACILITATE PLACEMENT OF GIRDERS, THE CONTRACTOR MAY SET THE NO. 35 DOWELS BY ONE OF THE FOLLOWING METHODS:
1) SETTING DOWELS IN APPROVED NON-SHRINK GROUT IN PREFORMED HOLES HAVING A MINIMUM DIAMETER OF 75 mm.
2) SETTING DOWELS AT INITIAL CASTING OF CAP.

NOTE B

RISERS AT FIXED BENTS SHALL BE SLOPED TO APPROXIMATE TANGENT GRADE AT ℄ BENT. ELEVATIONS SHOWN FOR THESE RISERS ARE AT ℄ BENT.

CONNECTION DETAIL AT CONTINUITY DIAPHRAGM

DIAPHRAGM DETAILS

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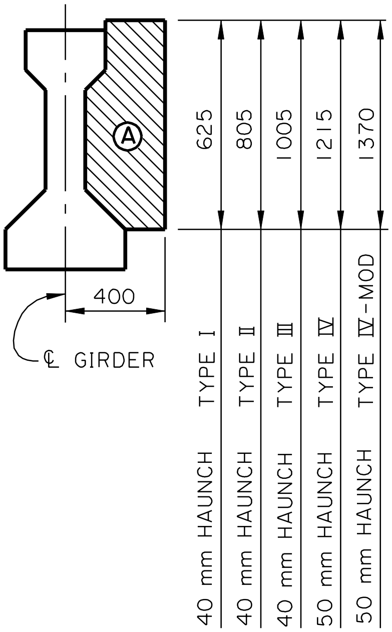
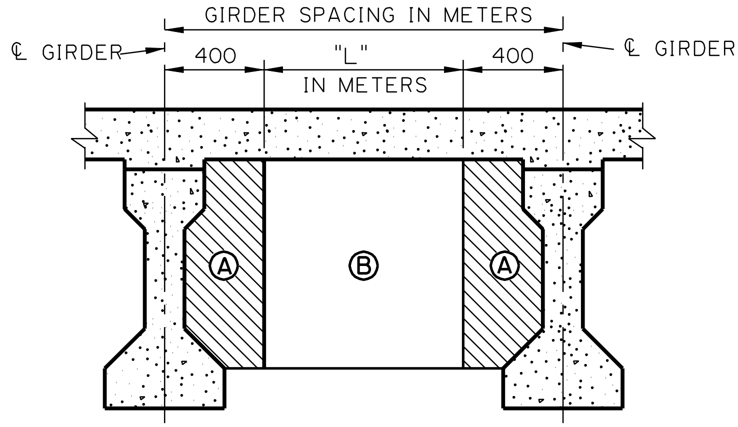
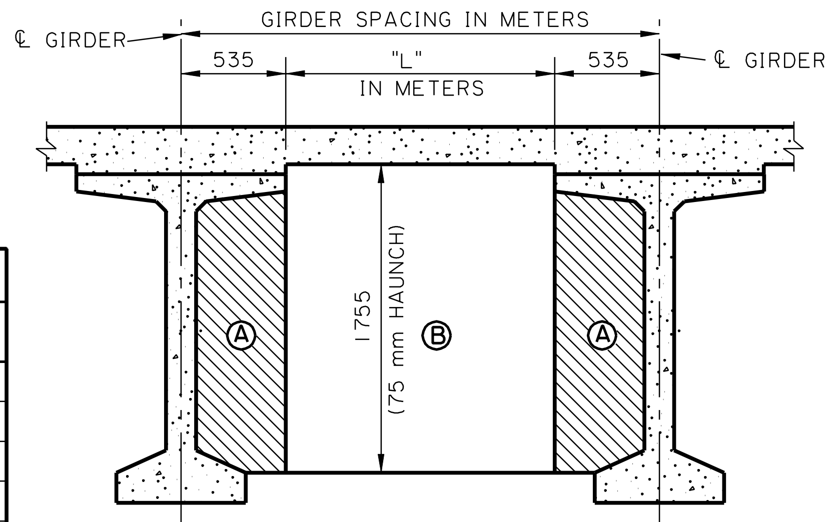




TYPICAL END DIAPHRAGM

(EXCEPT TYPE BT)



TYPICAL END DIAPHRAGM

(TYPE BT)

CONCRETE VOLUMES		
BEAM TYPE	VOLUME (A) m^3	VOLUME (B) m^3
I	0.0363	0.125 x "L"
II	0.0464	0.161 x "L"
III	0.0523	0.201 x "L"
IV	0.0575	0.243 x "L"
IV-MOD.	0.0620	0.274 x "L"
BT	0.1405	0.351 x "L"

TOTAL VOLUME = 2(A) + (B)

NOTE:

ALL DIMENSIONS AND QUANTITIES ARE FOR 90° CROSSINGS.

TYPE I-IV AND BT (CONCRETE COMPUTATIONS)

DIAPHRAGM DETAILS

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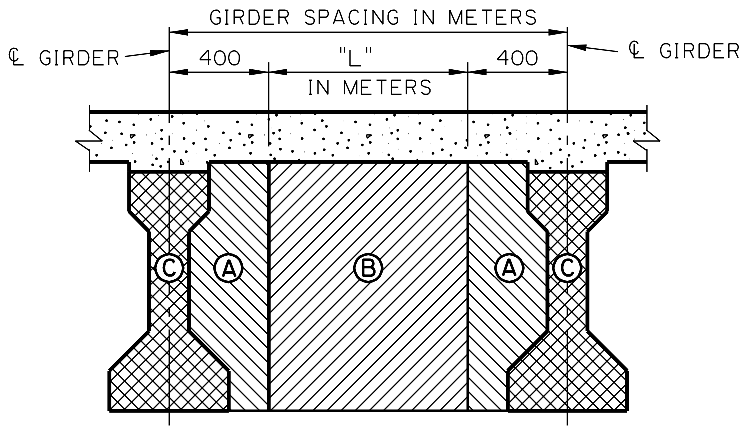
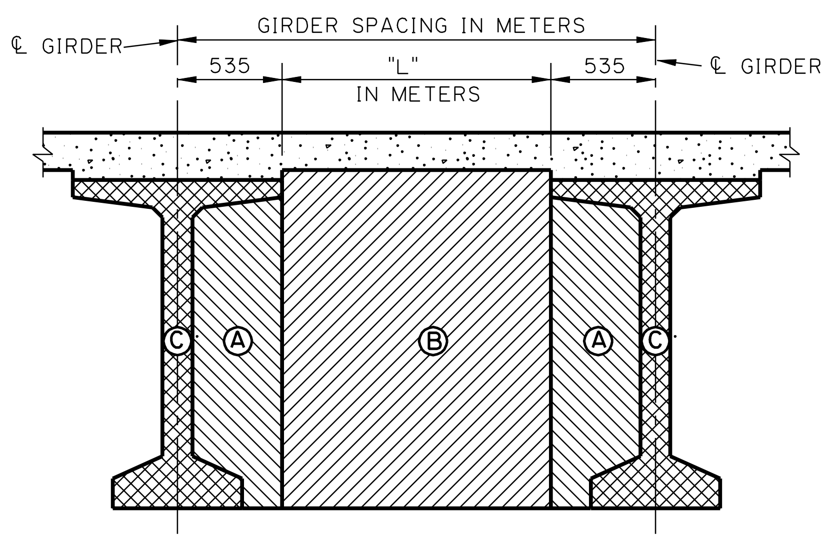


TYPICAL CONTINUITY DIAPHRAGM
(EXCEPT TYPE BT)

CONCRETE VOLUMES			
BEAM TYPE	VOLUME Ⓐ m^3	VOLUME Ⓑ m^3	VOLUME Ⓒ m^3
I	0.0514	0.1875 x "L"	0.0265
II	0.0644	0.2388 x "L"	0.0355
III	0.0708	0.2963 x "L"	0.0548
IV	0.0754	0.3550 x "L"	0.0761
IV-MOD.	0.0810	0.3938 x "L"	0.0879
BT	0.1834	0.4763 x "L"	0.0737

TOTAL VOLUME AT BENT = (2Ⓐ+Ⓑ) * NO. OF BAYS + (Ⓒ * NO. OF GIRDERS)



TYPICAL CONTINUITY DIAPHRAGM
(TYPE BT)

TYPE I-IV AND BT (CONCRETE COMPUTATIONS)

DIAPHRAGM DETAILS

Rev. 12/01/98

GENERAL GUIDELINES FOR STEEL SPANS

ANALYSIS

1. Load factor design shall generally be used except when horizontally curved girders are involved.

2. Horizontally curved girders will be designed in accordance with the latest edition and interim of the "AASHTO Guide Specifications for Horizontally Curved Highway Bridges". Where diaphragms are connected to the bottom flange, they shall be bolted only. Cross frames will be primary members. A note shall be placed in the plans referencing the additional camber requirements for heat curving as noted in the supplemental specifications.

3. When less than **three (3)** longitudinal girders (plate or box girders) are used in a structure, the members shall be designated as fracture critical. The portions of the girder to be designated as fracture critical are the top and bottom flanges in tension and webs. Also, the longitudinal stiffeners, gusset plates and all connections welded longitudinally to the web in the tension area of the web are considered fracture critical. The limits of all fracture critical plates shall be designated on the plans.

4. In general, simple and continuous steel spans shall be designed for composite action. The exception would be in a main girder-floor beam-stringer system when the stringers are non-composite because of incompatibility with main girder.

5. Continuous girders designed as composite will generally be assumed non-composite in the negative moment regions, i.e., the reinforcing steel will not be assumed to act composite with the girder. No shear connectors will be placed in these regions; however, the **1%** minimum longitudinal reinforcing requirements of the 1996 LFD AASHTO Specifications sub sections 10.38.4.3 and 10.50.2.3 are still applicable.

6. A **600 N/m^2** dead load will be assumed for a future wearing course to be placed on the composite section.

7. Although different classes of steel have been used in steel girders (a typical hybrid would use higher strength steel in the flanges than in the web) for economy in the past, we have been moving away from this practice due to steel pricing and design considerations. Hybrid girders should be used only with the approval of the Bridge Design Engineer.

8. For projects that are designed for HST truck loading, it shall not be used for fatigue computations.

9. The curb and railing dead load will be placed on the composite section of the exterior girder only.

10. The steel section of exterior girders shall have at least the same structural capacity as the interior girders for future widening purposes.

11. For the design of friction type connections surface condition "A", (clean mill scale) will be assumed.

12. Structural steel may be A 709 Grades 250, 345 or 345W. Gusset plates, stiffeners, bracing and other secondary members are generally designed using Grade 250.

DETAILING

1. The haunch shall be set so there is no encroachment by cover splice plates and their connectors into the deck.

2. Cantilevered deck slabs shall have their bottom surface aligned with the bottom of the girder flange.

3. The flange widths shall preferably be specified in multiples of **50 mm.** Flange thickness will be specified in multiples of **5 mm** for **t** < **60 mm** and **10 mm** multiples for **t** ≥ **60 mm**.

4. The web plate heights shall be specified in multiples of **50 mm**. Web thickness will be designed as follows:

10 mm ≤	**t** ≤ **22 mm**	**2 mm** increments from **10 mm**
	t > **22 mm**	**5 mm** increments from **25 mm**

5. The location of "permissible welded shop splices" in girder web and flange plates shall be shown on the girder sheets. Generally, the limit of plate lengths without splices is **15 m.**

6. Full penetration groove welded connections at transitions in thickness or in width shall have slopes no steeper than 1 to 5.

7. When applicable, the following note shall be added to the plans:

   THE CONTRACTOR MAY PROPOSE ALTERNATE SPLICE LOCATIONS FROM THOSE SHOWN IN THE PLANS, OR BOLTED FIELD SPLICES IN LIEU OF WELDED SPLICES, ALL AT NO ADDITIONAL COST TO THE DEPARTMENT AND SUBJECT TO THE APPROVAL OF THE ENGINEER PRIOR TO INCLUSION IN THE SHOP DRAWINGS.

8. Field splice locations are generally in low moment areas or where a section change is planned. Member lengths ≤ **35 m** or weigh ≤ **40 000 kg** are two approximate maximums for pieces that can be handled efficiently either in the shop or at the site. When the girder is erected over a road open to traffic, consider locating the field splice outside of the traveled lane to minimize disruption of traffic.

9. A minimum plate thickness of **10 mm** shall govern for both primary and secondary structural members. Unstiffened webs are generally more economical for webs approximately **1250 mm** deep or less; above that, consider a partially stiffened web.

10. When required, transverse stiffeners shall generally be placed on one (1) side of the web except at cross-frames, and shall be preferably on the inside of exterior girders normal to flanges.

11. The longitudinal stiffener shall normally be placed on the opposite side of the web from transverse stiffeners, and cut **$4t_w$ to $6t_w$** (**50 mm**) short of any transverse stiffeners it would intersect with.

12. Special attention must be paid to welding details to avoid critical fatigue condition. For longitudinal stiffeners, fillet weld in the compression zone, but bolting may be required in the transition compression-tension zone.

13. Transverse stiffeners shall be cut **$4t_w$ to $6t_w$** short of the tension flange, and seal welded to the compression flange. In areas where both flanges are subject to tension stress, welding to both flanges will generally be acceptable if the appropriate fatigue category is used in the design of the flange. Stiffeners at cross-frames and floorbeams shall be full depth and fillet welded to both the top and bottom flange, and the proper fatigue category shall be used.

14. Bearing stiffeners shall be groove welded or milled to bear and fillet welded, and shall be seal welded on the top. They shall be set vertical over the bearing device.

15. For structural steel welding, the minimum weld sizes shall be in accordance with the AWS Specifications. All items relating to welding shall be in accordance with the latest ANSI/AASHTO/AWS Bridge Welding Code.

16. The following information shall be included for steel spans:

    a) Moments, shears, and reactions values should be provided at critical points for each loading stage.

    b) The vertical curve ordinates, dead load deflections for the various load stages, and cambers should be shown either on a diagram or in tabular form.

    c) Cambers for composite girders shall take into account creep by calculating long term deflection based on 3n section properties.

    d) The above deflection information shall be shown at the span $^{1}/_{10}$ points for continuous spans and ¼ points for simple spans.

17. In general, bolted connections shall be made with **ASTM A 325M** bolts. **ASTM A 490M** bolts will not be used.

18. Shear connectors shall normally include an alternate for both **19 mm** and **22 mm** diameter studs, and the height shall be such that there is a minimum of **50 mm** penetration in the slab.

19. On long girders, deflections cause excessive longitudinal movements at the bottom of the girder ends. The designer must account for these movements when designing the girder bearings.

20. Seismic loading must be accounted for. Since Louisiana has an acceleration coefficient of **4.0%** and both importance classification result in a seismic performance category equal to "A", minimum analysis is required. Structures must meet minimum seat width and the connection requirements between the superstructure and substructure.

21. Inspection Details: All projects involving steel spans with more than **9 m** of vertical clearance above the ground shall provide details for inspection access. For grade separation structures with web depths ≥ **1800 mm** this will normally consist of a Class A galvanized wire rope **12.7 mm** in diameter (1 x 7 with breaking strength ≥111 kN) run through holes in the transverse stiffeners or clip angles and fastened at each span end. Wire ropes shall be located on both sides of the interior girders webs and the inside of the exterior girders. The rope should be placed about 1.2 m from the bottom flange. On major river crossings, inspection cables will be required on both sides of all girders on the main spans. More elaborate details will be required and will be handled on an individual basis. Minimum items to provide would include an enclosed access ladder at every pier and a walkway along the face of the cap.

22. Shop Assembly of Structural Steel: All construction plans for bridges involving structural steel shall have a general note as follows: "Shop assembly shall be by the Progressive Truss or Girder Assembly Method in accordance with Subsection 807.18 except that for structures less than **50 m** long or less than four contiguous panels, assembly shall be by the Full Truss or Girder Assembly Method.

23. Tack welds, which are not remelted, by welding required on design drawings, will not be allowed.

24. Stress relieving may be necessary for those applications where weldments must retain dimensional stability during machining (such as bearings). Contract drawings or specifications shall indicate welded assemblies to be stress-relieved by heat-treating. Finish machining shall be done after stress relieving[5].

25. All main load carrying steel members require Charpy V-Notch Impact Test (CVN), as per 1013.01 of the Standard Specifications. In order to comply with this requirement, the plans are to show the location and length of those members or portions of members subject to tension and which require CVN testing. Fracture critical members are a more critical subset of load carrying members. See the FCM section later in this chapter.

26. Use lateral bracing only if required.

27. The end of the girder and the bearing stiffeners shall be vertical. Intermediate transverse web stiffeners may be either all normal to flange or all vertical.

28. A pouring sequence will be shown dividing the deck into segments which minimizes deck cracking by allowing construction joints near contraflexure points and by loading positive moment areas first. Each pour must be completed in 4 hours with a maximum rate of 45 m^3 per hour. (Unless a higher rate is approved by the Bridge Design Engineer for special cases.)

29. The camber diagram shall include a note, which states: Cambers shown are for pouring sequence shown. If the contractor wishes to use a different sequence which meets the requirements of minimizing deck cracking and 4 hour set times, he shall submit his sequence with its associated camber diagram to the bridge design engineer for approval prior to the first shop drawing submittal.

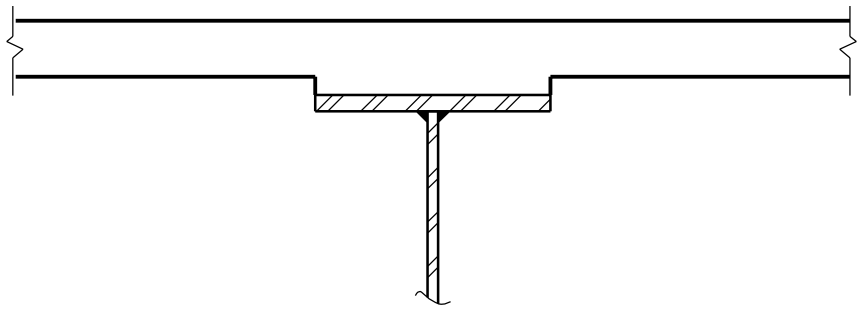
30. Unless otherwise specified, an approved three-coat waterborne paint system shall be used for coating all new metal surfaces requiring painting. See special provisions, such as E-85 (9/94 p. 494) from the contracts section and QPL 68 for further information.

[5] see 1988 AWS D 1.5, section 4.4

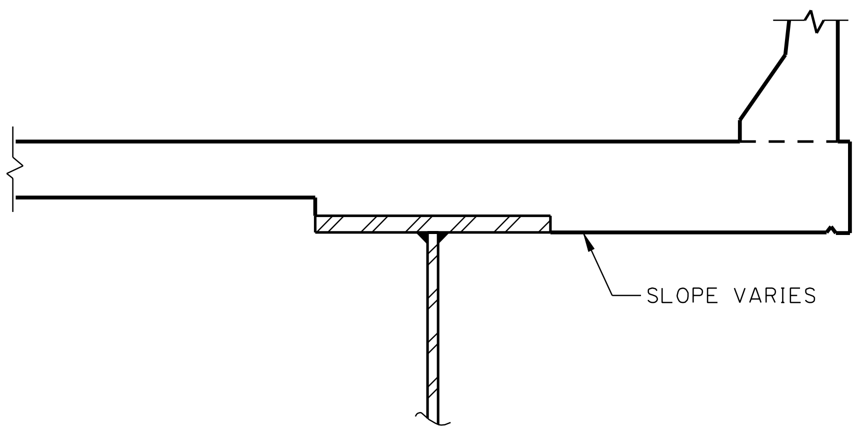
31. Swing Spans are not considered to be fracture critical by the bridge design section. Load case II for swing spans shall use temperature difference between top and bottom chords of **15°C** and a short span uplift equal to **25 mm**. *Dead load plus impact* shall be added to case II worst wedge reaction for total case II design loads.

32. All bolting for structural steel bridge members shall utilize Direct Tension Indicator (DTI) washers during installation. When DTI s are used, the bolts are tightened until the bolt tension collapses the washer protrusions to the specified 127 µm. gap. This will ensure proper bolt tension. Inspection is accomplished by means of a 127 µm. Feeler gage inserted to the washer gap.

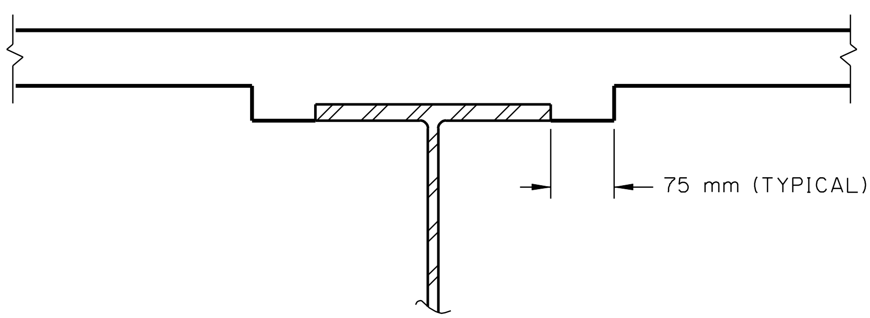
    The DTIs will always be galvanized with the additional requirement that for use on unpainted weathering steel, DTI shall be epoxy coated to prevent accelerated sacrificial galvanized coating loss.



INTERIOR PLATE GIRDER (COMPOSITE DESIGN)

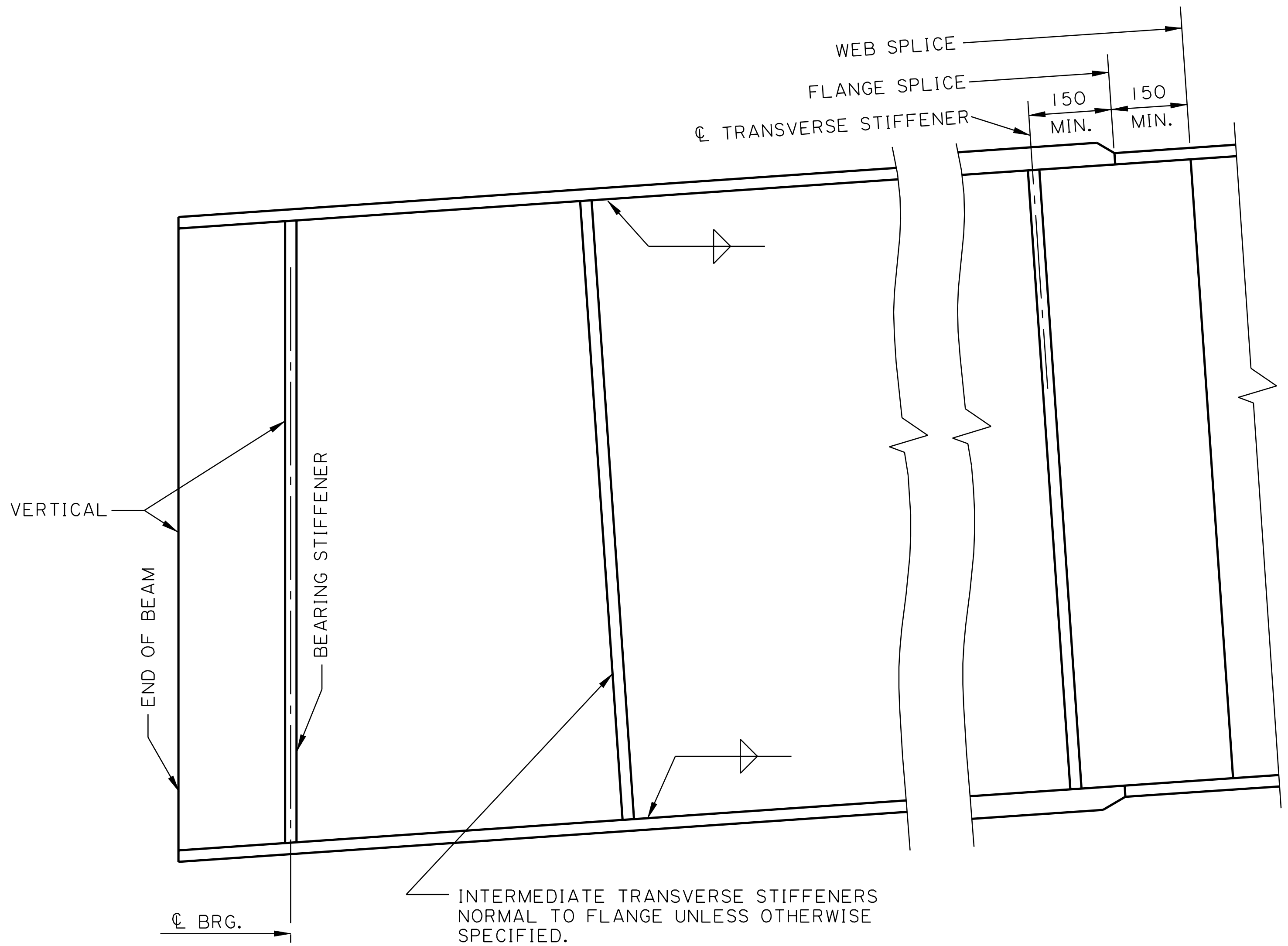


EXTERIOR PLATE GIRDER (COMPOSITE DESIGN)



INTERIOR STRINGER (NON-COMPOSITE)

TREATMENT OF SLAB AND HAUNCH ON STEEL GIRDER STRUCTURES



ELEVATION - TRANSVERSE STIFFENERS

STIFFENERS

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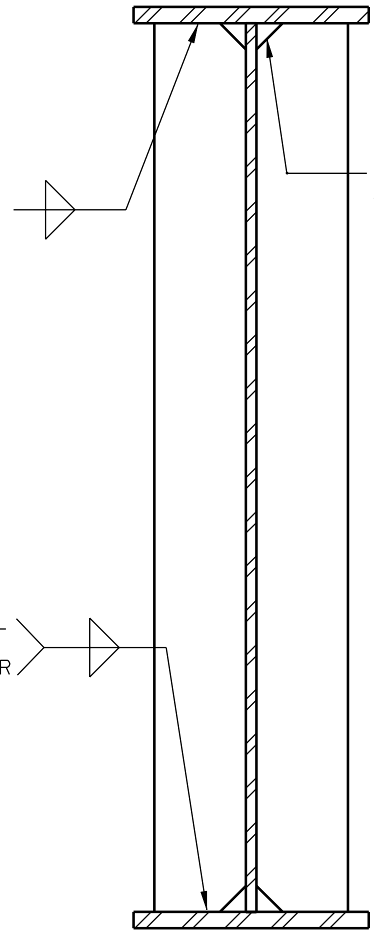
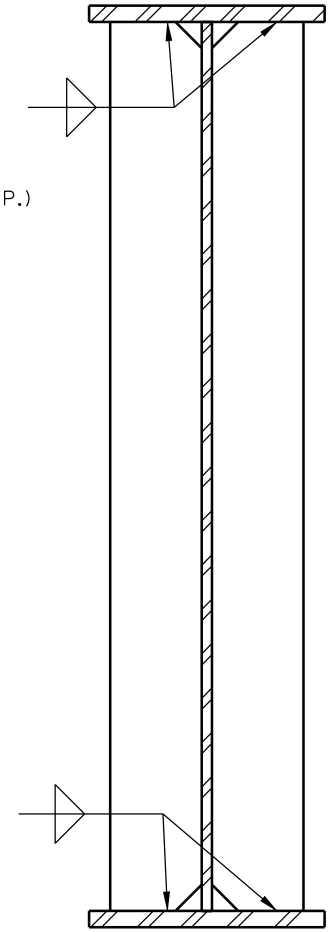
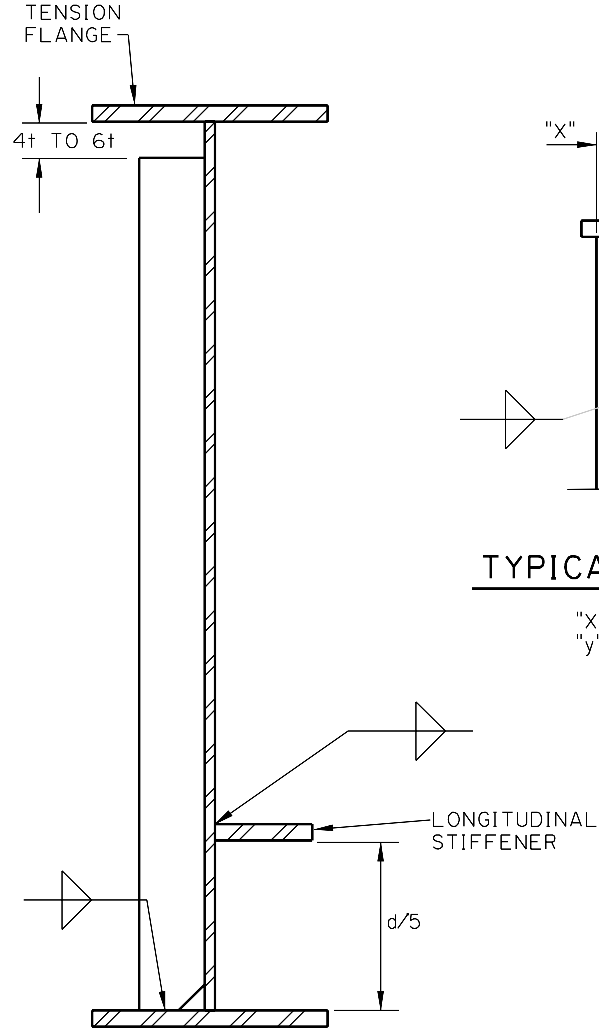


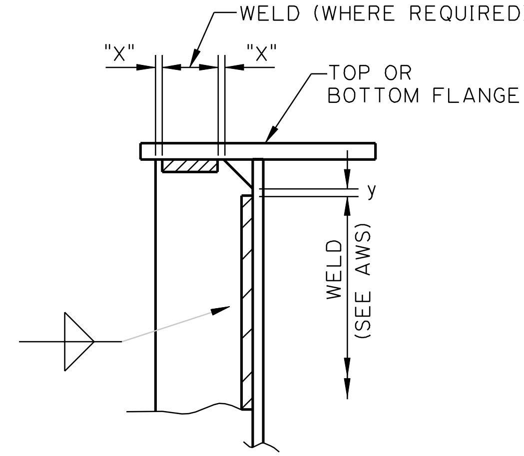
BEARING
STIFFENERS



CROSS-FRAME
STIFFENERS



TRANSVERSE
STIFFENERS



TYPICAL WELDING DETAIL

"X"= 6 mm ± 3 mm
"y"= 12 mm ± 6 mm

TYPICAL SECTIONS THRU GIRDER

STIFFENERS

2 OF 4

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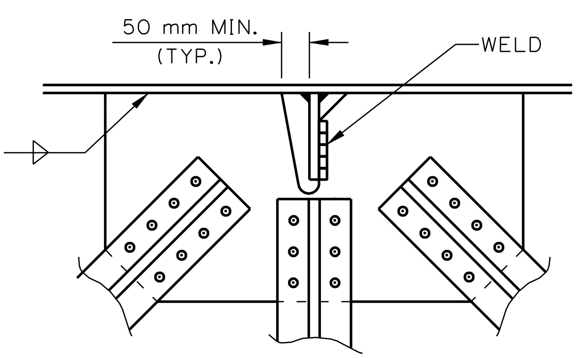
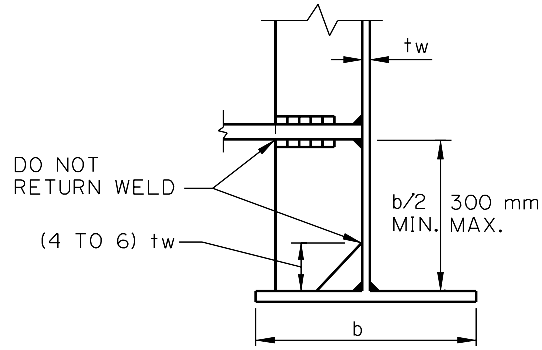
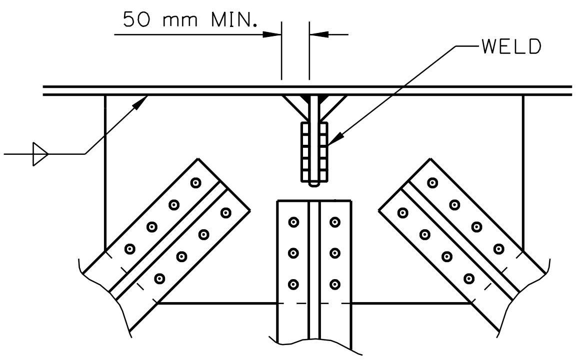
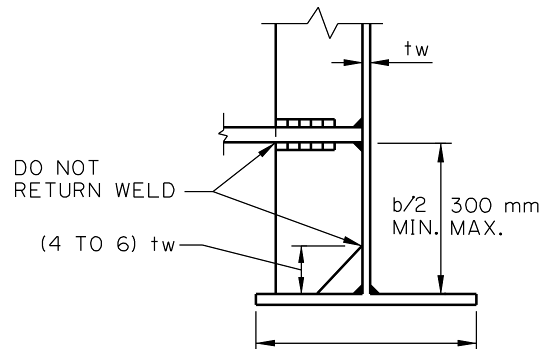


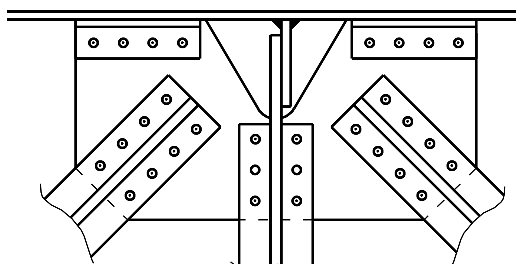


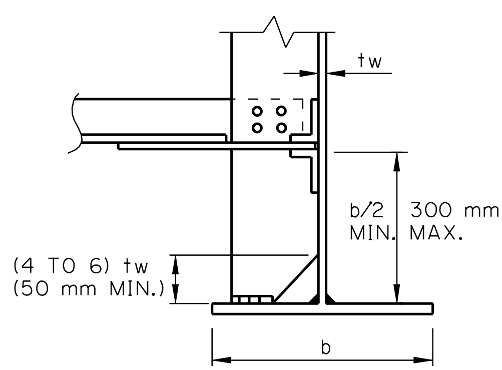
LOW LATERAL
LOAD DETAIL





WELDED HIGH LATERAL
LOAD DETAIL





BOLTED HIGH LATERAL
LOAD DETAIL

(PARTICULARLY TO BE USED WITH
SKEWED SPANS OR CURVED GIRDERS)

LATERAL CONNECTION PLATE

STIFFENERS

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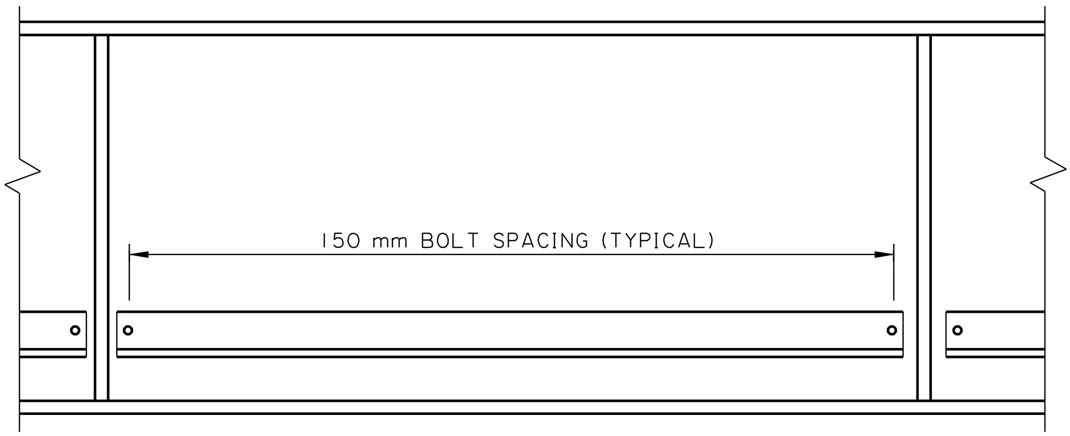
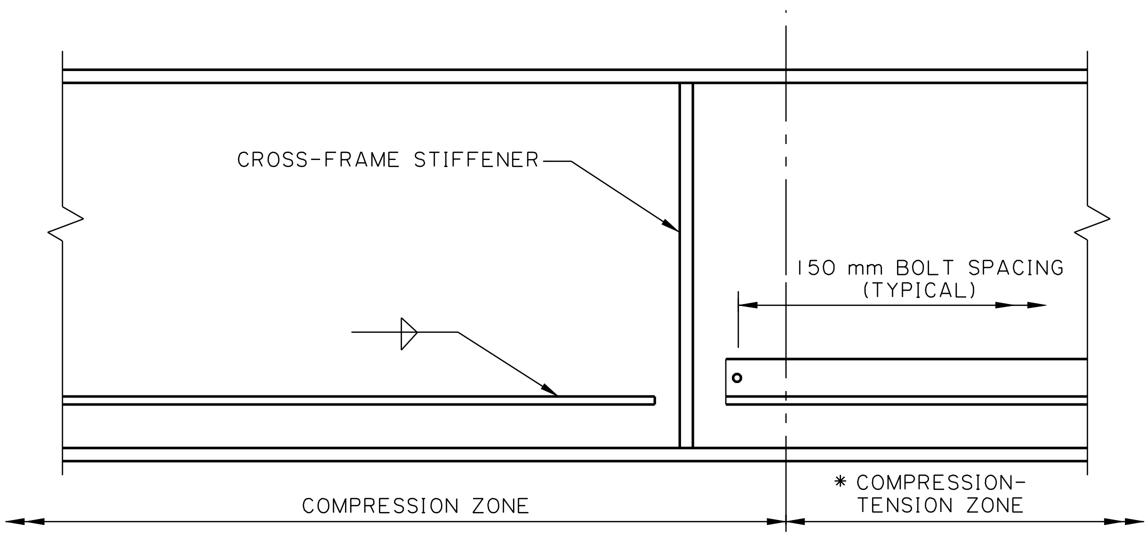


DETAIL SHOWING LONGITUDINAL STIFFENER IN COMPRESSION-TENSION ZONE



DETAIL SHOWING TRANSITION FROM WELDED TO BOLTED LONGITUDINAL STIFFENER

NOTE:

IF ANY PORTION OF THE LONGITUDINAL STIFFENER BETWEEN CROSS-FRAME STIFFENERS WILL BE IN TENSION, THAT SECTION SHOULD BE BOLTED.

ELEVATION - LONGITUDINAL STIFFENERS

STIFFENERS

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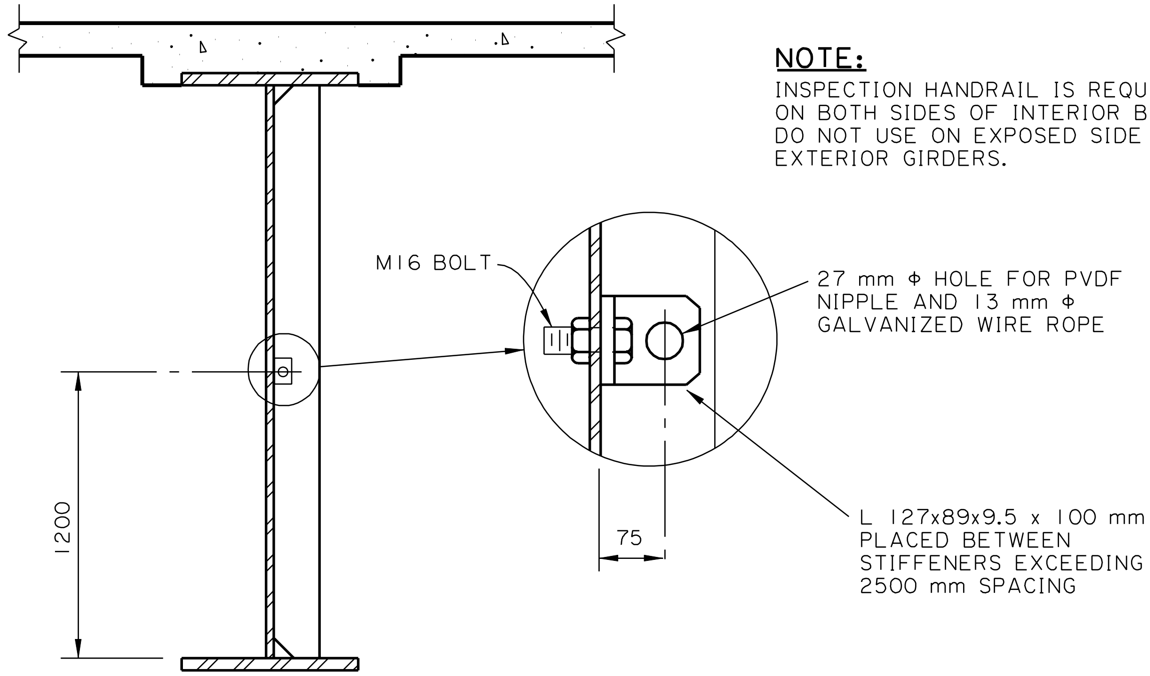
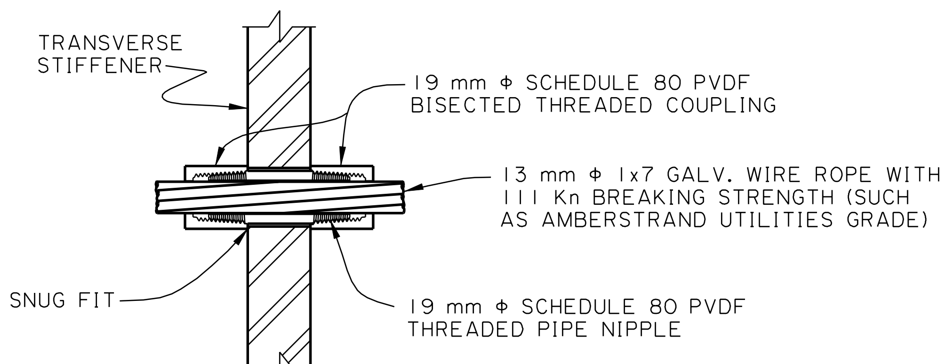
NOTE:
INSPECTION HANDRAIL IS REQUIRED ON BOTH SIDES OF INTERIOR BAYS. DO NOT USE ON EXPOSED SIDE OF EXTERIOR GIRDERS.



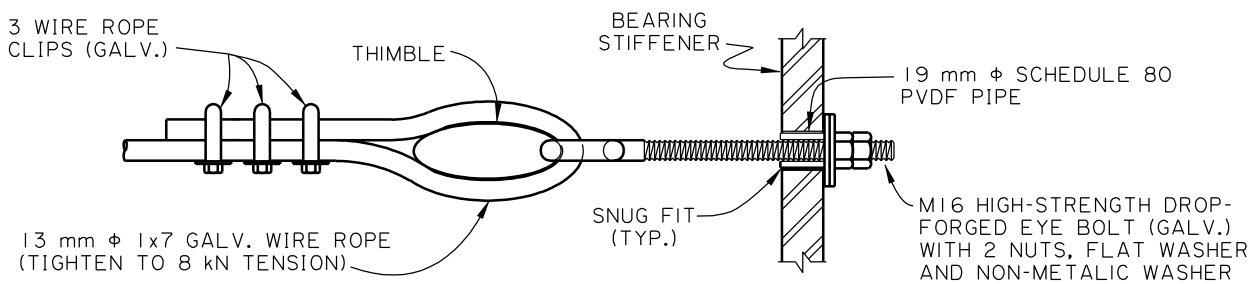
TYPICAL GIRDER SECTION

(SHOWING WIRE ROPE LOCATION)



DETAIL OF SAFETY CABLE BUSHING

(SHOWING WIRE ROPE PASSING THROUGH STIFFENER)



DETAIL OF SAFETY CABLE TERMINATION

(OPPOSITE END MAY UTILIZE PIN ANCHOR SOCKET)

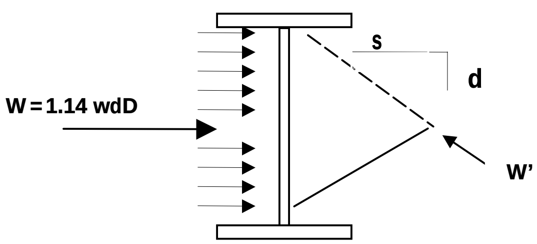
INSPECTION HANDRAIL

Cross Frames Diaphragms

1. Cross frames have a high price per kilogram but are given less attention by the designer than primary members.

2. Cross frame spacing is based on stability requirements during erection, lateral torsional buckling of compression flange and 1996 AASHTO LFD 10.20.1. However, 1996 AASHTO LRFD 6.7.4 allows the engineer to determine the need for permanent cross frame based on engineering analysis rather than arbitrary specified requirements. It may be economical to increase dimensions of the flange and reduce the number of cross frames to be fabricated to achieve a cost-effective design. The Bridge Design Engineer must approve the spacing greater than 7.6 m.

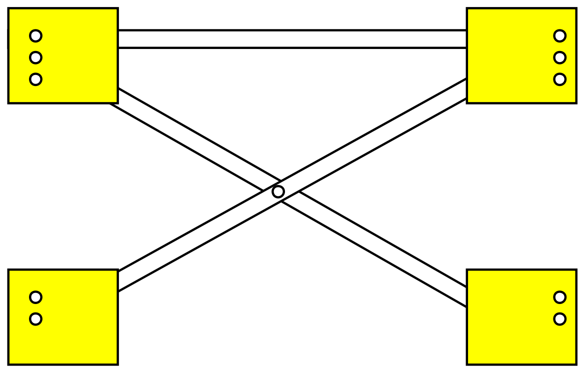
3. Unless required due to horizontal curve, treat elements of a cross frame as secondary members (AASHTO 10.20.1) to reduce undesirable load paths and design according to articles 10.7.1, 10.19.1.2 and 10.20 . Once the deck is placed, cross frames provide little resistance to wind loads but still may be required in negative moment regions.



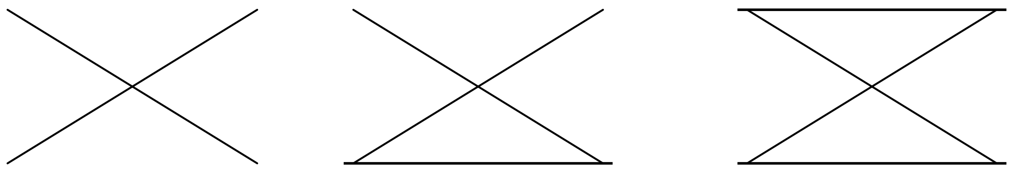
W = wind load (Newtons) on cross frame
$w = 2400\ N/m^2$
d = beam depth (meters)
D = cross frame spacing
s = girder spacing (meters)
W' = axial load in cross frame member in Newtons

$$= \frac{1.14 \times w \times (d) \times D \times (s^2 + d^2)^{1/2}}{2s}$$

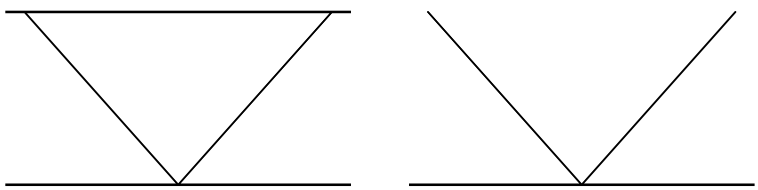
4. Reduce the cost by not mixing connections within a cross frame. Construction requires welded joints to be welded all around unless otherwise noted, then painted. In a bolted joint, faying surfaces receive two coats of paint before assembly. If a bolted joint is used in the middle, with welded ends, construction requires all surfaces to be painted before welding, then painted again. Avoid this mix.

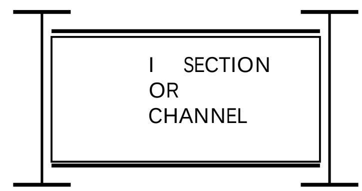


DIAPHRAGM CONFIGURATIONS



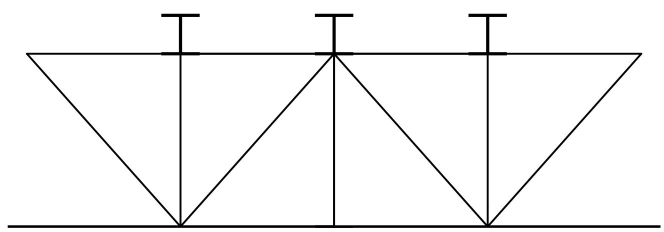
"X" FRAMES ANGLES OR TEES



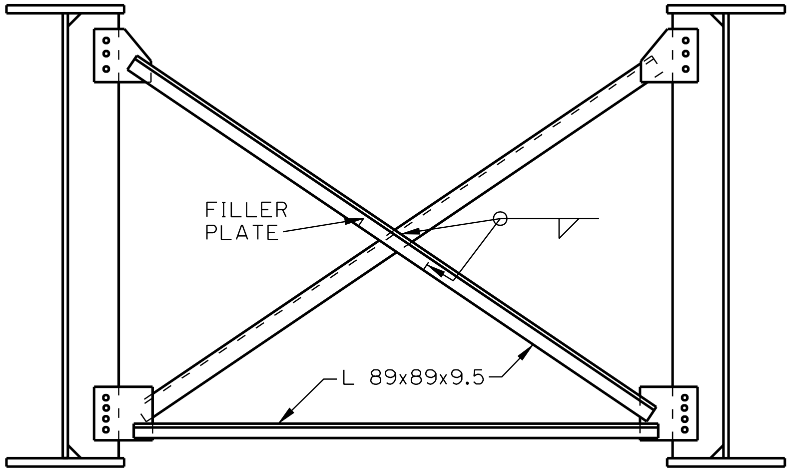


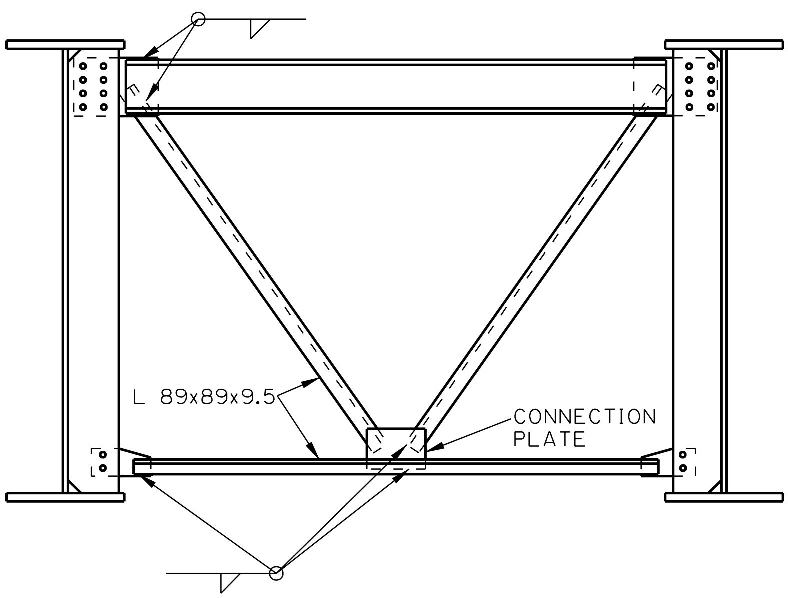
"K" FRAMES ANGLES OR TEES

SOLID



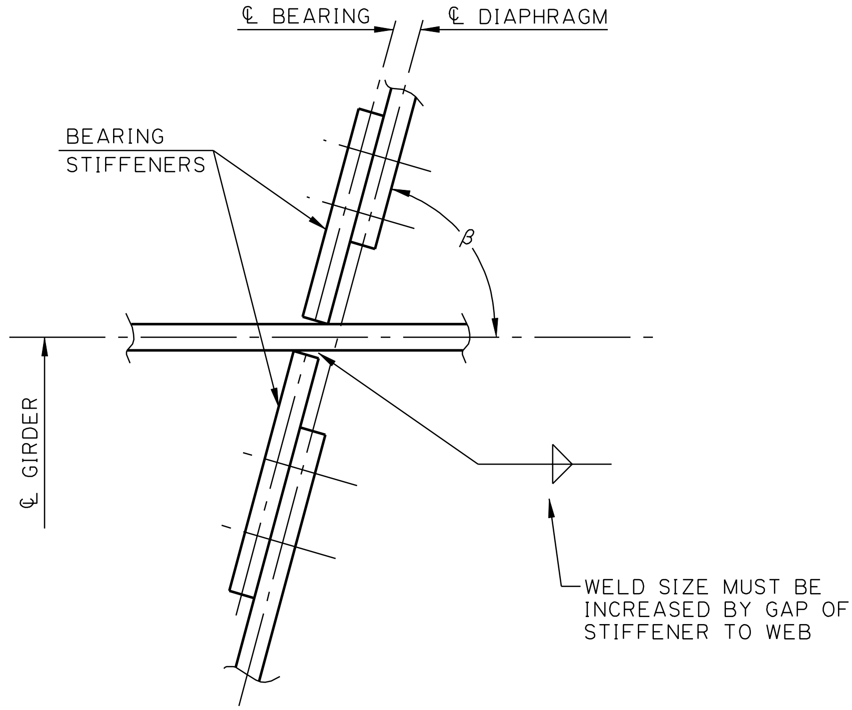
STEEL SUBSYSTEM   ANGLES OR TEES



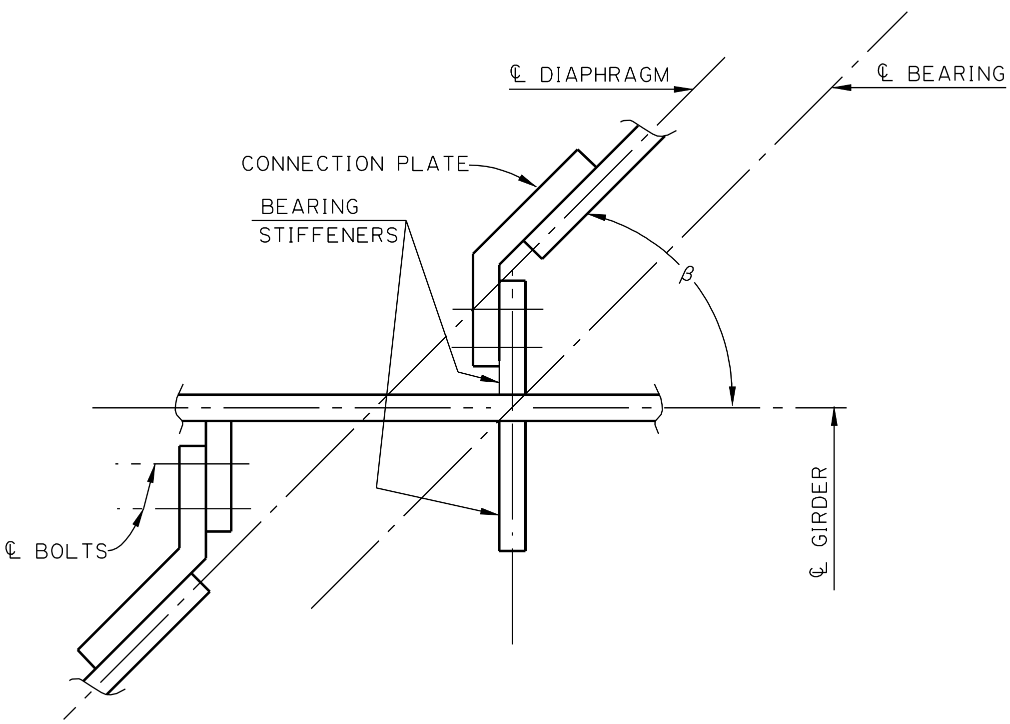


TYPICAL DETAILS

CROSS-FRAMES



β = 90° TO 75°



β UNDER 75°

TYPICAL SKEWED DIAPHRAGM CONNECTIONS

TYPICAL SKEWED

STEEL GIRDER OPTIMIZATION

1. Fabrication costs money. Only about 35% of the total fabricated cost is material. Economy can be realized only when fabrication is minimized. Select fabrication details that are the least complex and yet will be able to perform the required function. Practice design duplication, which means that the greater the number of elements and details on a given job that are replicated, the lower the fabrication costs, will be.

2. Whenever possible, use standard specifications for both bridge material and fabrication.

3. Optimum girder design is more sensitive to web thickness and number of stiffeners than to depth the web.

4. High strength steels result in shallower optimum depths and more economy than grade 250 steel.

5. For girder webs up to 1700 mm in depth, economy favors designs that require no intermediate transverse stiffeners to satisfy shear requirements. It is recommended that 8.0 kg web material be saved for every kilogram of transverse stiffener material added to the structure. Longitudinally stiffened girders do not become economical until girder depths exceed 2.5 m.

6. In general, the greatest economy for multiple girder bridges can be realized by using the least number of girders consistent with a reasonable deck design.

7. Always specify a component with the simplest details that will serve the intended function.

8. Approximately 360 kg. (span $>$ 30 m) or 270 kg. (span $\leq$ 30 m) of flange material must be saved to justify the introduction of a shop flange splice. Normally, the most economical design results when the flange (tension and compression) size is carried through the entire positive moment field section. Generally two flange splices will be justified in each of the flanges in negative moment field sections.

FATIGUE DESIGN OF STEEL STRUCTURES

Introduction

This article covers fatigue design for all types of steel bridges: simple span, continuous spans, composite and non-composite designs. The mechanics of fatigue design, such as the number of load cycles, fatigue category and allowable range of stress shall be as specified by the AASHTO Specifications. This article is to amend and supplement the AASHTO Specifications in accordance with current Department policy.

Commentary

Bridge fatigue has become a matter of increasing concern in recent years. Details used for attachments to steel structures have been reviewed and modified in an effort to eliminate known fatigue problem areas. The designer should select details that allow a higher stress range. This can be done by using Category "A" through "D" details in lieu of Category "E" and "F" details; bolting attachments instead of welding; and using redundant members instead of non-redundant members. Additional background information is available in the American Institute of Steel Construction "Bridge Fatigue Guide Design and Details" by Dr. John W. Fisher.

Analysis and Details

1. Cover plates shall generally not be used; however, when there are no other practical options, the plate termination should be located as close to the girder end as possible.

2. Generally, Category "E" and "F" details are not permitted.

3. Return welds at the ends of plates shall be avoided.

4. Butt welds in longitudinal stiffeners must be full penetration and radiographically checked. This is an important connection, because if it fails, it will cause cracking in the web details that will radiate through the flange.

5. Do not use details that bring triaxial welds together, such as at the juncture of a gusset plate and a stiffener. Welds should be stopped short of intersections.

6. Welds for transverse stiffeners generally are stopped short of the flange, and are not returned.

7. Stiffeners at floor beam, cross frame or diaphragm connections, should be welded to both flanges to prevent out of plane distortion of the girder web.

8. Fatigue shall be checked only for MS-18 truck or lane loading as applicable.

9. Unless otherwise directed by the Bridge Design Engineer, the two main girders and the pivot girder on swing span bridges are not designed as FCM.

FRACTURE CRITICAL MEMBERS

Introduction

This article applies to steel structures, which contain fracture critical members. A fracture critical member is one whose failure would lead to collapse of the entire structure. Current AASHTO Specifications do not contain information on fracture critical members and are supplemented herein in accordance with current Department policy and proposed AASHTO Specifications.

Commentary

Fracture critical members (FCM) are those tension components of a bridge whose failure would be expected to result in the collapse of the bridge. Tension components of a bridge consist of pure tension members or flexural members that are subject to tension stress. Any attachment having a length in the direction of the tension stress greater than **100 mm** that is welded to a tension component of a fracture critical member shall be considered part of the tension component and, therefore, shall be considered "fracture critical". All welds in fracture critical members are subject to the welding requirements contained in the fracture control plan.

Identification

Definition of Fracture Critical Member: These are members that will cause a major collapse or failure of the structure should they fail. The following descriptions explain how to identify the critical areas of fracture critical bridges. In general, the critical areas are the high-tension zones where only a few separate steel elements are available to carry the load.

Two-Girder Systems

Welded steel bridges with only two main girders have a fracture critical area at the lower flange in the midspan regions and at the upper flange in the region over each continuous support. The upper flanges near the end supports do not constitute fracture critical areas.

Box Girder Bridges, Single Box Design

The single box design of a steel box girder bridge is a fracture critical bridge. The bottom plate is welded to two or sometimes three web plates, each of which has a flange plate welded to its upper edge. The bottom plate will be the fracture critical area at the midspan region of each span. The upper flange plates will be fracture critical in the region over each continuous support except where the plans show a substantial amount of additional reinforcing steel parallel to the flanges. In this case, the additional reinforcing steel provides redundancy, guarding against a catastrophic failure.

Steel Caps

Where supports cannot be located directly under the bridge, bridge designers sometimes use a steel box beam cap to span between widely spaced columns. The superstructure is then supported in the midspan region of the cap. In these cases, there is one fracture critical area at the lower flange in the midspan region. If the box beam cantilevers over a supporting column, the top flange over the column is fracture critical.

Truss Bridges

When a truss bridge has only two main trusses it may contain fracture critical members. The fracture critical members would be single-element tension members such as an eye bar or an I-beam tension member. Tension members consisting of multiple separate steel elements are not FCM. Each fracture critical tension member would count as one fracture critical area.

Suspended Span Bridges, Two-Girder Systems

The same criteria apply that were discussed for two-girder systems. In addition, the pins and hangers, which support the two-girder suspended span, are considered fracture critical.

Analysis

When practical, the designer should select a type of structure that eliminates or reduces the use of fracture critical members. Generally, this may be done by use of redundant members. The designer shall determine which components are fracture critical and determine the limits of the members, which are fracture critical.

Details

1. The contract plans shall clearly delineate the components which are fracture critical members and which components require Charpy V-Notch input testing as per 1013.01 of the Louisiana Standard Specifications. This information may be designated on the girder elevation view or shown on a separate sketch. The limits of tension or reversal stress areas must be clearly dimensioned.

2. All welding procedures must be approved prior to the submitting of shop drawings.

3. Shop drawings shall show the location and extent of fracture critical members. Weld symbols shall indicate the welding procedure to be used.

4. To aid future inspections, fracture critical members shall be identified in tabular form on the first sheet of the bridge plans such as:

IDENTITY OF FRACTURE CRITICAL MEMBERS		
DESIGN SHEET NO.	TYPE OF MEMBER	LOCATION WITHIN MEMBER
115	Main girder	Lower flange
123	Cap	Top flange and web
126	Cross frames	Upper horizontal strut

GUIDELINES FOR WEATHERING STEEL DESIGN

1. Steel (ASTM A 709 GR 345W) will not develop a protective oxide coating if it remains wet for a long time (greater than 60% of the time) or is contaminated by salt from any source.

2. The most common cause of corrosion problems is caused by runoff water leaking through the deck and wetting the diaphragm, the girders, and the bearings in vicinities of joints.

3. Weathering steel is resistant only to certain specific types of atmospheric corrosion in a limited range of environments. Adequate periods of drying are needed without prolonged periods of wetting. Air must not contain excessive corrosion contaminants. Corrosion penetration exceeding **7.5 μm/yr./surface** will prevent oxide coating from forming.

4. Approval from the Bridge Design Engineer for use south of US 190 will be required.

5. General considerations in weathering steel design:

   a) Prevent water from ponding.

   b) Divert deck runoff water from steel superstructure.

   c) Prevent debris accumulation that traps moisture.

   d) Avoid environments that would cause salt contamination.

   e) Make decks continuous where feasible and use drain troughs where continuity is not feasible.

   f) Use integral abutments whenever possible.

   g) Welding should be with electrodes matching corrosion resistance of the weathering steel.

   h) Bolts should have corrosion resistance equal to or exceeding the weathering steel such as ASTM A 325M Type 3; however, galvanized bolts should not be used.

   i) Load indicator washers are not recommended, as they may create crevices.

   j) Connections should be designed so as not to entrap water (drain holes in gusset plates, etc.). If this cannot be avoided, painting must protect contact surfaces between plies.

k) Near white blast cleaning is recommended for surfaces for public view and commercial blast cleaning is satisfactory for other surfaces.

l) Weathering steel bridges are not maintenance free. Periodically removing debris with compressed air may be needed.

m) Rehabilitation may require repainting of corroded areas. Paint may require additional specifications in consideration of difficulty of achieving near white surfaces. Paint coverage (due to pitting and roughness) may be as little as one fourth that shown on the manufacturer's product data sheet.

n) The girder at the expansion bearings will be painted for a distance of 1.5 times the girder depth. (Paint color to match final oxidized surface color.)

6. Girders

a) Minimize the number of horizontal surfaces on which water can pond or debris can accumulate.

b) Minimize the number of re-entrant corners.

c) Use details which are self-cleaning and self-draining.

d) Avoid crevices.

e) For members with thickness less than 38 mm an additional thickness of 0.8 mm per exposed surface should be added to compensate for the normal corrosion.

7. Box members and girders

a) Should be constructed tight if feasible (members or orthotropic deck girders).

b) Where tight construction is not feasible, provide adequate hatches or vents to provide a draft.

c) Inside surfaces of box girders may be left bare, however, where inaccessible to inspection, inside should be painted.

d) Inside surfaces of box or tubular members should be painted with an extra heavy coating around openings.

REFERENCE: National Cooperative Highway Research Program (NCHRP) Report 314

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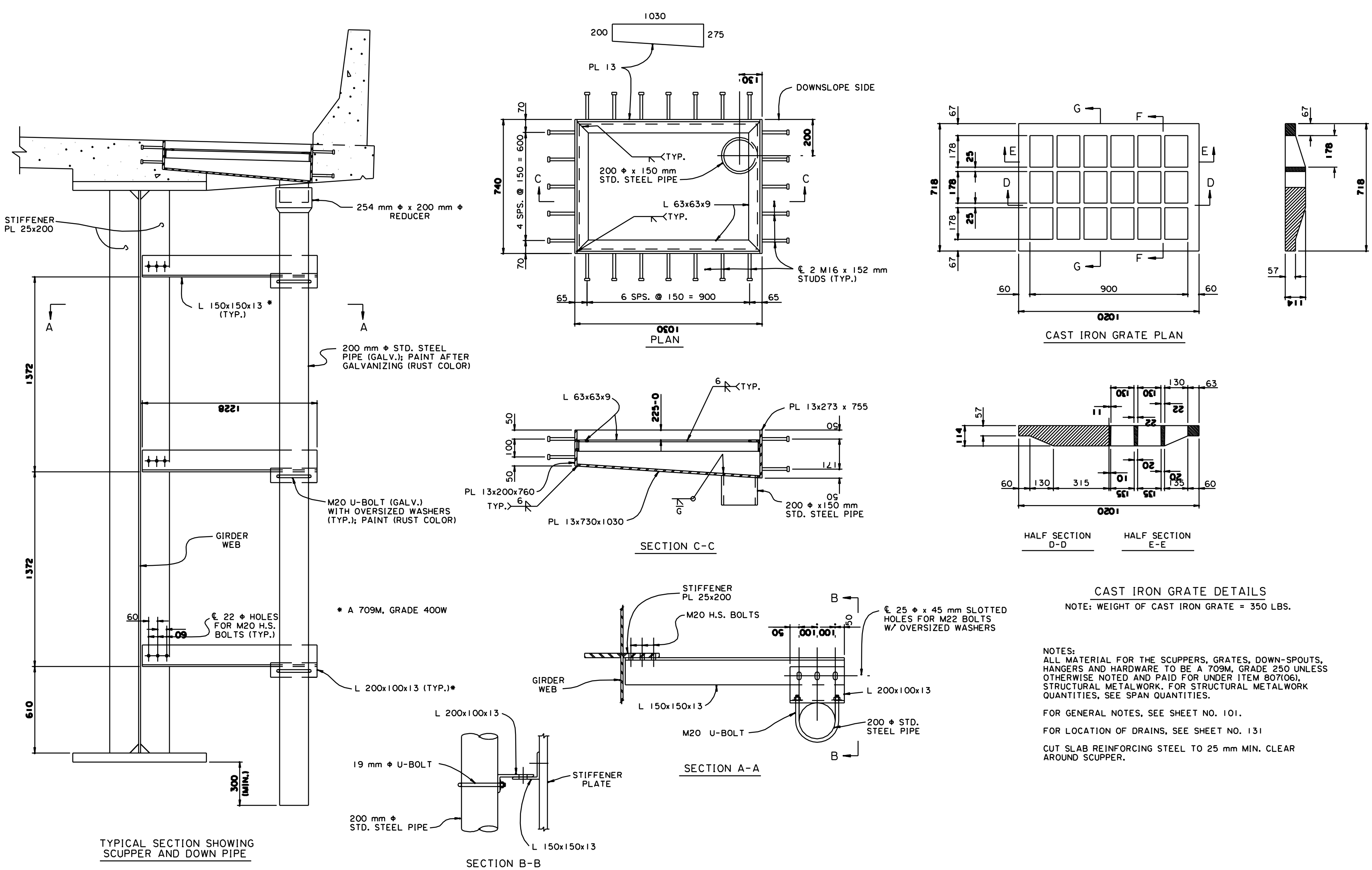


CAST IRON GRATE DETAILS

NOTE: WEIGHT OF CAST IRON GRATE = 350 LBS.

NOTES:
ALL MATERIAL FOR THE SCUPPERS, GRATES, DOWN-SPOUTS, HANGERS AND HARDWARE TO BE A 709M, GRADE 250 UNLESS OTHERWISE NOTED AND PAID FOR UNDER ITEM 807(06), STRUCTURAL METALWORK. FOR STRUCTURAL METALWORK QUANTITIES, SEE SPAN QUANTITIES.

FOR GENERAL NOTES, SEE SHEET NO. 101.

FOR LOCATION OF DRAINS, SEE SHEET NO. 131

CUT SLAB REINFORCING STEEL TO 25 mm MIN. CLEAR AROUND SCUPPER.

NO.	DATE	REVISION DESCRIPTION	BY

DESIGNED / CHECKED / DETAILED / CHECKED / DATE / SHEET

BRIDGE AND STRUCTURAL DESIGN

EXAMPLE
SCUPPER DRAIN DETAILS

PARISH / FEDERAL PROJECT / STATE PROJECT / SHEET NUMBER

BEARINGS

The most common bearing system currently being used by the Department is the neoprene bearing system. Neoprene pads are used exclusively for all prestress girder designs and are often used for steel spans where lengths are not prohibitive. As outlined in AASHTO, elastomeric bearings should be of uniform thickness (risers sloped to match grade), with plain pads using 70 and reinforced pads using 60 durometer hardness material. For reinforced bearings, Louisiana uses only steel reinforcement with **3 mm** minimum edge cover. Neoprene pads for prestress girders shall be in accordance with the miscellaneous span and girder details and will be used on all standard projects without additional design effort.

For longer steel spans, other type bearings are used. Teflon sliding bearings may be used, if approved by the Bridge Design Engineer. The rocker bearing is the most common type and should be designed in accordance with the AASHTO Specifications. All large bearings shall be stress relieved. A fully supported pin between the upper and lower saddles should be used in order to prevent bending of the pin which could result in localized high stresses, galling, popping and disintegration. For long span deep girder designs, significant movements can be generated from span deflections and must be accounted for in the bearing design. The use of lead materials for bedding the steel bearings to the bearing surfaces is prohibited.

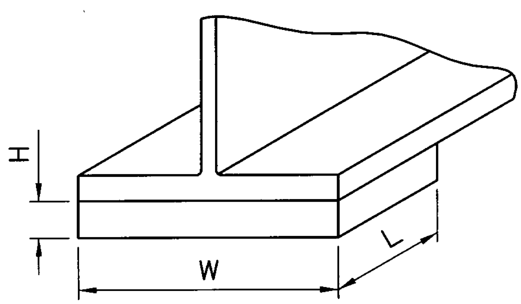
On horizontally curved girder spans the expansion bearings should be oriented on a line projected from the fixed bearing to the expansion bearing. If finger joints are used, the fingers should also be oriented along this line.

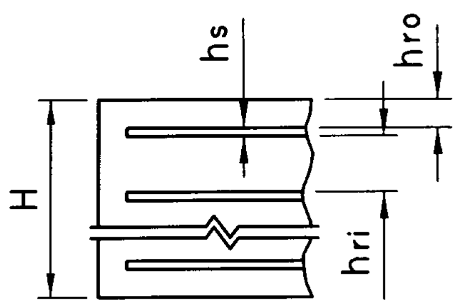
RECTANGULAR NEOPRENE BEARING DESIGN

<u>Required Data</u>

Girder: lower flange width (b_f) =
dead load reaction (P_{dl}) =
live load reaction (P_{ll}) = w/o impact
expansion length (l) = distance from fixed bearing to expansion bearing being designed

<u>Given Data</u>





ITEMS / CONSTANT	CONCRETE	STEEL	REFERENCE
T (C °)	39 °C	68 °C	3.16
α (^{1}/C°)	10.8×10^{-6}	11.7×10^{-6}	8.5.3.10.2.2
SHRINKAGE	208.33×10^{-6}	104.17×10^{-6}	BDM, chapter 3

PAD / VARIABLE		PLAIN	REINFORCED	REFERENCE
HARDNESS		70	60	SHORE "A"
$F_cTL(N/mm^2)$		10.8×10^{-6}	1.38	14.4.1.1
G	Gc	1.38	0.90	T 14.3.1
	Gs	2.07	1.38	T 14.3.1
B		1.8	1.4 outer	14.2
		1.8	1.8 inner	14.2
CREEP		45%	35%	T 14.3.1

Design (Method A)

$W \approx b_f$ =

$TL = (P_{dl} + P_{ll})$ =

$L \geq TL \div (F_{cTL} \times W)$ =

$\sigma_c = TL \div (L + W)$ =

1. $h_{ri} \leq G_c \times (L * W) \div [2 \times \sigma_c \times (L + W) \times \beta]$ = compression 14.4.1.1

$h_s = 1.6$ mm Std Spec 1018.14(a)
$F_y = 227.5$ N/mm^2 Std Spec 1018.14(a)

2. $F_{all} = 0.55\, F_y$ T10.32.1A

3. ≤ 165 N/mm^2 T10.3.1A, 2 million cycles

4. 2) $h_{ri} \leq (F_{all} \times h_s) \div (1700 \times 4.448/25.4)$ = shim strength 14.4.1.6

5. ≤ 16.9 mm

6. $\Delta h = [\propto(\Delta T) + \text{Shrink}] \times l$ = DOTD & FHWA use total _T

7. $\Delta s \approx$ _h = Δh modified for pier flex.

8. n = No. of inner layers =

3) $h_{ri} \geq (2\times\_s) \div (n + 2/1.4)$ = shear 14.4.1.3

4) select h_{ri} and n to satisfy equation 1,2,& 3

h_{ri} =

n =

select $h_{ro} \approx h_{ri}/1.4$ =

check stress: $h_{crit}/\beta \geq h_{ri}/1.0 \geq h_{ro}/1.4$ =

$S/\beta = (W*L) \div [2(W + L)(h_{crit}/\beta)]$ =

$\sigma_c < G_c(S/\beta)$ =

5) $h_{rt} = n \times h_{ri} + 2 \times h_{ro}$ =

$h = h_{rt} + (n+1)*h_s$ =
$\leq L/5$ (plain) = stability check 14.4.1.5
$\leq L/3$ (reinforced) =
6) $H = G_s(L \times W)(\Delta s/2) \div h_{rt})$ slip check 14.6.1
$\leq 0.2P_{dl}$ if no slip apparatus is used

7) $\varepsilon_{ci}^{TL} = f(S/\beta,\sigma_c)$ = See figure F14.4.1.2B or

$$= \left[\frac{\sigma_c}{2.3(\frac{S}{\beta})^{1.63}} \right]^{\left(\frac{1}{1.15+0.0286(\frac{S}{\beta})} \right)}$$

International Bridge Conference 88-17 Eq 37

≤ 0.07FHWA Region 3 SCEF Comm.10
$\Delta_c^{TL} = n \times h_{ri} \times \varepsilon_{ci}^{TL} + 2 \times h_{ro} \times \varepsilon_{co}^{TL}$
$\leq$? mm agency instantaneous deflection limit

$\Delta_c^{DL} = n \times h_{ri} \times \varepsilon_{ci}^{DL} + 2 \times h_{ro} \times \varepsilon_{co}^{DL}(1+creep)$ agency long term deflection limit

≤ 3 mm

GD = grade difference between riser and bottom of girder

1. $\theta^{TL} = (\theta^{DL} + \theta^{LL}) \approx w_{DL}L^3/24EI + 1/400 \pm GD$

CORROSION PROTECTION METHODS

GENERAL

Details given in Chapter V provide adequate corrosion protection for the average application. For those locations where severe corrosion potential exists (exposure to coastal splash zones, deicing chemicals, areas where there is a history of corrosion problems), the Bridge Design Engineer will determine the method to protect the concrete reinforcing steel, including direct treatments (galvanizing or epoxy coatings), corrosion inhibitors (calcium nitrite), and silica fume or fly ash for reducing concrete permeability.

Fly ash

A pozzolan is a by-product of pulverized coal-fired electric power generation. It is physically smaller than the average cement particle and is round.

PROS:

1) Improves workability (pumping and ease of flat-work finishing) of fresh concrete
2) Reduces peak temperature of mass concrete
3) Long term reaction products help fill in spaces between hydrated cement particles, thus lowering permeability to water and aggressive chemicals
4) Reduces required water in mix for given slump
5) Increases long-term strength of concrete if moist environment and moderate temperatures maintained
6) Low cost
7) Allows reduction in cement content

CONS:

1) Reduced early strength, increases setting time (both initial and final)
2) Decreases entrained air
3) Class C can cause corrosion (use Type F)

COSTS:
From 3.3 to 4.4 cents per kilogram (1995) with maximum rate of 25% of the cement by weight.

Microsilica

A by-product of the production of silicon and ferrosilicon alloys. It, like fly ash, is a pozzolan, but with a much higher silicon dioxide content and much smaller, rounder particle sizes. It is more reactive than fly ash. Use the densified form.

PROS:

1) High early strengths

2) Significantly reduces permeability

CONS:

1) Requires use of high water reducing admixtures to maintain slump

2) Requires use of admixtures to obtain desired entrained air

3) Increased plastic shrinkage, due to less bleed water, requires surface to be covered to prevent rapid evaporation

4) More expensive than fly ash

COSTS:
$1.30 per kilogram (1995) with maximum rate of about 10% of the cement by weight.

Calcium Nitrite

A corrosion inhibiting liquid added to the concrete mix that reacts with embedded steel to inhibit chemical action of chlorides on the reinforcing steel. Corrosion initiation is delayed and corrosion rates are controlled by determining the anticipated chloride ion content of the concrete over the design life of the structure and adding sufficient quantity to counteract that value. Recommended rates are from 10 to 30 liters per m^3.

PROS:

1) Offers built-in protection that can be adjusted for service life required

2) Only proven corrosion inhibitor with enhanced mechanical properties of concrete

CONS:

1) Price

COSTS:
$1.85 per liter (1995)